THE

IMPROVEMENT OF RIVERS.

A TREATISE ON THE METHODS EMPLOYED FOR IMPROVING STREAMS FOR OPEN NAVIGATION, AND FOR NAVIGATION BY MEANS OF LOCKS AND DAMS.

BY

B. F. THOMAS,

United States Assistant Engineer, Member of the American Society of Civil Engineers,

AND

D. A. WATT,

United States Assistant Engineer, Member of the American Society of Civil Engineers.

FIRST EDITION.

FIRST THOUSAND.

NEW YORK:

JOHN WILEY & SONS.

LONDON: CHAPMAN & HALL, LIMITED.

1903.

ROBERT DRUMMOND, PRINTER, NEW YORK.

PREFACE.

THE following treatise has been prepared in the hope that it may assist in meeting a want which has probably been felt by many engineers and others engaged on the improvement of rivers. With the exception of De Lagréné's "Cours de Navigation Intérieure," published in 1873, and which is now out of print, there is no work known to the authors which treats of this important subject except in a general way, or with sufficient detail for those engaged on actual construction or design. Much valuable information is to be found scattered through the various Government Reports relating to inland navigation, and matter of great interest has also appeared from time to time in various domestic and foreign publications. A search through these for information on any special point would, however, involve an expenditure of time and labor which few of those engaged in active work could give, even if the documents were accessible to them. Moreover, since the appearance of De Lagréné's work many new methods have come into use, especially in America, and wider experience has been gained, and as experience is probably more necessary for successful work in this branch of engineering than in any other, owing to the imperfect understanding of the laws governing the flow of rivers, the authors believe that a treatise combining the results of theory and recent modern practice may prove of some utility in this field. They have accordingly endeavored to include all the important points of design and construction which are likely to be met with in ordinary practice, and the calculations have been simplified as far as possible so as to bring them within the range of those who do not possess a thorough technical education. It is hoped, therefore, that the book may prove of use not only to engineers, but also to inspectors, surveyors, and others who are engaged on the more practical side of work.

The authors desire to acknowledge their indebtedness to Brigadier-General G. L. Gillespie, Chief of Engineers, U. S. Army, and to the Officers of the Corps of Engineers, for their courtesy in granting access to drawings and data connected with the works under their charge and permitting the publication of certain of them, and also to return thanks to certain civilian engineers for information on similar matters.

Their thanks are also due to Major W. M. Black, Corps of Engineers, author of "The United States' Public Works," and to Mr. Edward Wegmann, author of "The Design and Construction of Dams," for permission to reproduce illustrations from

those works; and to the American Society of Civil Engineers and the Association of Engineering Societies for similar courtesies; and to the Commission for the canalization of the Elbe and Moldau in Bohemia (M. W. Rubin, Chief Engineer of Construction) for information and photographs of that system of improvements.

JULY, 1902.

LIST OF PRINCIPAL WORKS CONSULTED.

Cours de Navigation Intérieure. De Lagréné. 1873.
Rivières et Canaux. P. Guillemain.
Rivières à Courant Libre. F. B. DeMas.
Canalisation de la Moldau et de l'Elbe en Bohême V. Rubin.
Observations relatives aux barrages mobiles. De Lagréné.
Notice sur le barrage construit sur la Marne à Joinville. M. Malézieux.
Canalisation de la Meuse. Hans.
Barrages mobiles à grande chute. Boulé.
Amélioration du Rhône entre Lyon et la mer. Jacquet.
Amélioration des rivières à fond mobile par l'emploi des épis noyés. Jacquet.
Un nouveau système de barrage mobile. Janicki.
Barrage de la Mulatière. Also Avant-projet-général pour l'amélioration du Rhône. Pasqueau.
Barrage de Suresnes, et canalisation de la Seine entre Paris et Rouen. Boulé.
Étude sur les murs des réservoirs. Krantz.
Manuel de l'Ingénieur des Ponts et Chaussées. Debauve.
Mémoire sur divers barrages en course d'exécution en France. Caméré.
Note sur les divers moyens employés pour améliorer les conditions de navigabilité des rivières. Janicki. (Translated by Col. Wm. E. Merrill, U. S. A.)
Annales des Ponts et Chaussées, various numbers.
Le Génie Civil, various numbers.
Annales de la Construction, various numbers.
Die Rhein Regulierung. Krapf.
Vierter und fünfter Jahres-Bericht der Commission für die Canalisirung des Moldau- und Elbe-Flusses in Böhmen.
Handbuch der Ingenieurwissenschaft.
Deutsche Bauzeitung.
Journal of the Ministry of Transportation Routes (Russian).
Report of Board of Engineers upon the Applicability of Hydraulic Gates and Dams in the Ohio River. Col. Wm. E. Merrill, 1875.
The United States' Public Works. Capt. W. M. Black, U. S. A.
Reservoir Sites in Wyoming and Colorado. Capt. H. M. Chittenden, U. S. A.
Mitering Lock-gates. Lieut. Harry F. Hodges, U. S. A.
Report of the Board of Engineers, Commission on Deep Waterways from the Great Lakes to the Atlantic.
Civil Engineering (Report on Paris Exposition of 1883). Wm. Watson.
Civil Engineering (Report on Vienna Exposition, 1873).
Rivers and Canals. L. F. Vernon-Harcourt.
Improvement of the Maritime Portions of Rivers. (Same author.)
Civil Engineering. Patton.
Civil Engineering. Law and Burnell.
Civil Engineering. Rankine.

Masonry Construction. Baker.
Embanking Lands from River Floods. Wm. Hewson.
Embanking Lands from the Sea. John Wiggins.
Floods of the Mississippi. Wm. Starling.
Chicago Main Drainage Channel C. S. Hill.
Gauging of Streams. W. G. Price.
Manual of Irrigation Engineering. Also Irrigation in India. Herbert M. Wilson.
Water Power. J. P. Frizell.
The Design and Construction of Dams. Edward Wegmann.
Construction of Mill-dams. Jas Leffel & Co.
Theory and Practice of Surveying. J. B. Johnson.
Zur Schiffbarmachung der Flüsse. Schlichting.
Inland Waterways (Russian). Zbrogek.
Desfontaine's Movable Dams (Russian). Timonoff.
History of Embanking. Dugdale.

Also various numbers of the following:

Annual Reports Chief of Engineers, U. S. Army.
Annual Reports U. S. Geological Survey.
Transactions American Society of Civil Engineers.
Proceedings Engineers' Club of Philadelphia.
Proceedings Engineers' Society of Western Pennsylvania.
Proceedings Institute of Civil Engineers.
Journals of the Association of Engineering Societies.
Transactions Civil Engineers' Society, Cornell University.
Journals of the Franklin Institute.
Engineering News.
Engineering Record.
Scientific American.
Van Nostrand's Eclectic Engineering Magazine
Engineering.
The Engineer.
Engineering Magazine.

CONTENTS.

PART I.

General Characteristics—Preliminaries to Improvement—Surveys.

PART II.

Improvement of Open Rivers.

CHAPTER I.

REMOVAL OF BARS AND OTHER OBSTRUCTIONS.

CHAPTER II.

REGULARIZATION.

CHAPTER III.

DIKES AND THEIR EFFECTS.

CHAPTER IV.

PROTECTION OF BANKS.

CHAPTER V.

LEVEES.

CHAPTER VI.

STORAGE RESERVOIRS.

CHAPTER VII.

IMPROVEMENT OF RIVER OUTLETS.

PART III.

Improvement of Rivers by Canalization.

CHAPTER I.

GENERAL DESIGN AND CONSTRUCTION OF WORKS FOR CREATING SLACK WATER.

CHAPTER II.

LOCKS.

CHAPTER III.

LOCK GATES AND VALVES.

CHAPTER IV.

FIXED DAMS.

CHAPTER V.

MOVABLE DAMS.

CHAPTER VI.

NEEDLE DAMS.

CHAPTER VII.

CHANOINE WICKET DAMS.

CHAPTER VIII.

GATE CURTAIN AND BRIDGE DAMS, SHUTTER AND A-FRAME DAMS.

CHAPTER IX.

DRUM WICKETS AND BEAR-TRAPS.

CHAPTER X.

RECENT CONSTRUCTION OF MOVABLE DAMS.

APPENDIX A.

Dimensions, Etc., of Locks and Dams on Certain Rivers.

APPENDIX B.

Specifications.

APPENDIX C.

Laws for the Protection and Preservation of the Navigable Waters of the United States.

ERRATA.

Page x, Index, Ch. VII, last line, *for* Dimensions of Dams *read* Dimensions and Cost of Dams

" 33, formula $Qcl = (h + \frac{2}{3}d)$, etc., should read $Q = cl(h + \frac{2}{3}d)$, etc.

" 97, line 26, *substitute* leads *for* lead

" 129, top line, *for* depth . . . of 12 feet *read* depth to 12 available feet

" 144, 4th par., 1st line, *for* reefs and bowlders *read* snags and bowlders

" " 4th par., 2d line, *for* \$300 per mile *read* \$800 per mile

" 146, foot-note, *for* 1790 *read* 1798, and *for* This lock has been carefully preserved *read* It was destroyed in 1814 by the United States troops, but the old sills and floor still remain. The lock was restored some years ago as a curiosity.

" 153, lines 9 and 15, for R sin 2α and R cos 2α read T sin 2α and T cos 2α

" 161, line 23, *for* use them *read* use the cribs

" 165, " 27, should read $\frac{1}{10}$

" 179, " 24, for $T \sec \alpha = \frac{\rho'}{2 \sin \alpha} \times \sec \alpha$, etc.,

read $T \cos \alpha = \frac{\rho'}{2 \sin \alpha} \times \cos \alpha = \frac{\rho'}{2} \cot \alpha$

" 182, " 6, *after* wooden gates *add* The timbers were of Georgia yellow pine

" " " 10, *for* up to 18 feet *read* up to 15 feet

" 213, between lines 25 and 34 add v = observed velocity of approach

" 214, line 3, for V read v

" 250, *add to title of cut:* Dimensions in meters and centimeters

" 259, line 3, *for* last one *read* best known

" 263, title of cut, *for* Reversed Parker Type *read* Direct Parker Type. On the right hand side of section, at top of cut, is the legend "Up-stream Side"; this should read "Down-stream Side." In the "Details at A" is a bent piece marked "$\frac{1}{4}$" plate"; this should be "$\frac{1}{8}$" plate."

" 264, line 11, for $\sqrt{\frac{x}{1-n}} +$, etc., in formula read $\sqrt{\frac{x^2}{1-n}} +$, etc.

" 265, in table for $\frac{h}{H}$ = limit, 50° under Z, .173 should be .167

Drawing of Steel Gate, horizontal framing. In title, *change* moderate length *to* moderate lift. *Add the note:* The up-stream vertical cushion timber at the toe is frequently omitted, a vertical angle being then used to support the remaining piece.

LIST OF ILLUSTRATIONS.

PART I.

PART II.

PART III.

MOVABLE DAMS.

NEEDLE DAMS.

CHANOINE WICKET DAMS.

GATE, CURTAIN, AND BRIDGE DAMS.

SHUTTER AND A-FRAME DAMS.

DRUM WICKETS AND BEAR-TRAPS.

GENERAL VIEWS.

LIST OF DETAIL DRAWINGS.

THE IMPROVEMENT OF RIVERS.

PART I.

GENERAL CHARACTERISTICS—PRELIMINARIES TO IMPROVEMENT—SURVEYS.

CHAPTER I.

INTRODUCTION.

Progress of Navigation.—The utilization and improvement of watercourses is as old as history. The ancients improved and utilized streams and constructed canals, at first for irrigation, and later for navigation. Successive improvements have followed each other in the methods of navigation in order to keep pace with the requirements of commerce, and the methods of one hundred years ago would fail to satisfy the demands of the present, and one hundred years hence the methods now seemingly satisfactory will probably be considered wholly insufficient.

In early navigation trees were cut in the forest and rolled into the streams and floated singly to the point required for use. This was followed by assembling the logs in rafts held together by tie-poles pinned to the logs. Later, various kinds of merchandise and farm products were placed upon these rafts and transported, and this led to the introduction of small boats in which the merchandise could be more safely carried, and which were tied to the rafts. Finally, larger boats were built and floated independently, carrying considerable loads. In America a large amount of coal, farm products, tanbark, staves, etc., was taken to market in this manner. The boats were sold or wrecked for the material, upon arrival at destination, no ascending navigation being attempted.

Many of the rivers, however, afforded sufficient water for navigation only during the wet season, and this, together with the necessity of having power to turn the mills for grinding grain, carding wool, manufacturing lumber, etc., led to the construction of dams. These provided pools which were easy of ascent, and rivermen soon found that it was more advantageous to bring back their boats than to sell or cut them up,

and ascending navigation was accordingly undertaken. Here a great obstacle was encountered—the mill-dams were difficult to ascend, although sluices had been provided in their construction which could be opened and closed at will by means of small needles, or *poutrelles*. While the opening of these chutes was of great benefit to descending navigation, because of the increased depth and velocity it gave below, the very same causes detained ascending craft. This led to the introduction of locks, which were further beneficial in maintaining a good navigable depth in the pools, because of their economy in water consumption. But even this method, which for a time seemed perfectly satisfactory, failed to answer to increased commerce and the competition of railroads, and finally the introduction of movable in the place of fixed dams followed in France and other parts of Europe, and later in America.

Choice between Canal and River.—England's first engineer of canals, Brindley, who has been dead more than one hundred and twenty-five years, is credited with having said that "Rivers were created for the purpose of feeding canals," and acting on that principle he built a canal lateral to the Irwell and the Mersey. He was not alone in his belief, for many miles of canals were constructed both in Europe and America, along the side of streams which might have been utilized for navigation. The early experience of French engineers led them to believe that where a river was subject to high floods and carried much sediment, its improvement would be of uncertain value, as its bed would be liable to change and the lock entrances would silt up. Besides this, the velocity of the current, which would not exist in a canal, would always be more or less of a hindrance to ascending craft. The invention of movable dams, however, solved many of the problems which confronted engineers of that day, and while not all of them agreed that a lateral canal was preferable to a river, the preponderance of opinion was in that direction. Such canals are accordingly to be found on the Doubs, Scarpe, and other streams.

Locks.—The solution of change of level by locks is believed to have been made in 1439. The first one was used to assist in transporting marble to the Milan cathedral and was, according to Lombardini, built by Philip Visconti. Soon after their introduction all European countries made extensive improvements in waterways, and thousands of miles of canals were built. The Briare Canal, completed about 1642, and connecting the Loire and the Seine, in France, was the first summit-level canal, and was rendered possible by the invention of locks. The feasibility of thus connecting two distinct systems of waterways by means of canals and locks added a great impetus to internal navigation, and systems were constructed throughout France, Germany, Belgium, Russia, and later in England and the United States. There are to-day 3000 miles of canals and 4600 miles of improved rivers in France alone.

Ancient Canals.—It is believed that canals were constructed in China and Egypt long before the time of which we have authentic record. When subject to Rome Egypt furnished to her the necessary supplies, and during the reign of Menes, 2320 B.C., the course of the Nile was diverted by means of canals so as to drain and irrigate the pro-

ductive bottom lands. As far back as 1659 B.C., Sesostris opened canals for transporting merchandise, running at right angles to the Nile between Memphis and the sea. Julius Cæsar, Caligula, and Nero, each attempted to build a canal across the Isthmus of Corinth. The Mœris Canal, some 40 leagues in length, was finished about 1385 B.C., and conveyed water to a great lake during the wet season and returned it by an irrigation system during periods of drought. The Royal Canal of Babylon, constructed about 600 B.C., is one of the earliest of whose existence there is proof. The Canal of Marius, 102 B.C., connected the lower Rhone with the Mediterranean, while Alexander built a canal at Alexandria, 332 B.C., connecting the city with the Nile. The Tiber was cut to the sea by Claudius when the mouth of that river was blockaded by the enemy, and later emperors connected the sea with the interior by navigable canals. Charlemagne in the eighth century began a canal from the Rhine to the Danube, but it was not completed until the early part of the nineteenth century. This canal now connects the Atlantic with the Black Sea. In the eighth century a canal was built in China 650 miles in length. In order to overcome the changes in the level of this canal, the boats were drawn up or lowered down inclined planes, a plan not yet extinct.

American Canals and Rivers.—Under the advantages of river navigation many cities have been greatly developed, although much of their growth and prosperity preceded the introduction of works of improvement, which are matters of comparatively recent date. The invention of the movable dam was the chief factor in developing these conditions, although the invention of the lock four or five centuries earlier had rendered it possible to greatly improve navigation both in natural and artificial waterways. Rapid strides were taken in the canalization of natural watercourses in France upon the advent of the movable dam, and canals, which had become quite common, began to decline. In this country, however, river improvement was not actively undertaken until within a very recent period, and modern ideas for canalization have not yet secured that recognition their merits deserve, there being less than 100 miles of slackwater formed by movable dams and probably not more than 1000 miles of canalized rivers, all told, in the United States.

Prior to the invention of the locomotive and the construction of railroads, the State governments had given aid and encouragement to the establishment and maintenance of systems of artificial canals between the West and the seaboard. Millions of dollars were expended and hundreds of miles of canals were constructed and operated. These were mainly independent of the natural waterways and have been but poorly maintained, and in some cases, abandoned entirely. While the enormous development of the railway systems has caused a loss and a reduction in the operation of canals, there has been an increase of public interest in the rivers as avenues of commerce, and within the past twenty years the National Government has made extensive appropriations for the improvement and utilization of many important streams. However, no systematic plan applicable to several rivers forming an extended route of transportation has been adopted, each river having its own system of improvements.

The following statement is taken from a recent publication: "The first canal opened in the United States for transportation of passengers and merchandise was the Middlesex Canal, connecting Boston with the Concord River, in 1804. The following statement shows the cost of the principal canals of the United States used for commercial purposes:

Canals.	Cost of Construction.	When Completed	Length, Miles.	No of Locks.	Depth, Feet.	Location.
Albemarle and Chesapeake.	$1,641,363	1860	44	1	7½	Norfolk, Va., to Currituck Sound, N. C.
Augusta.	1,500,000	1847	9	.	11	Savannah River, Ga., to Augusta, Ga.
Black River.	3,581,954	1849	35	109	4	Rome, N. Y., to Lyons Falls, N. Y.
Cayuga and Seneca.	2,232,632	1839	25	11	7	Montezuma, N. Y., to Cayuga and Seneca lakes, New York.
Champlain.	4,044,000	1819	66	32	5	Whitehall, N. Y., to West Troy, N. Y.
Chesapeake and Delaware	3,730,230	1829	14	3	9	Chesapeake City, Md., to Delaware City, Del.
Chesapeake and Ohio.	11,290,327	1850	184	73	6	Cumberland, Md., to Washington, D. C.
Companys.	90,000	1847	22	1	6	Mississippi River, La., to Bayou Black, La.
Delaware and Raritan.	4,888,749	1838	66	14	7	New Brunswick, N. J., to Trenton, N. J.
Delaware Division.	2,433,350	1830	60	33	6	Easton, Penn., to Bristol, Penn.
Des Moines Rapids	4,574,950	1877	7½	3	5	At Des Moines Rapids, Mississippi River.
Dismal Swamp.	1,151,000	1794	29	7	6	Elizabeth River, Va., to Pasquotank River, N. C.
Erie.	52,540,800	1825	352	72	7	Albany, N. Y., to Buffalo, N. Y.
Galveston and Brazos.	340,000	1851	38		3½	Galveston, Texas, to Brazos River, Texas.
Illinois and Michigan.	7,357,787	1848	96	18	5½	Chicago, Ill., to LaSalle, Ill.
Illinois and Mississippi.	568,643	1895	4½	3	7	Around lower rapids of Rock River, Ill. Connects with Mississippi River.
Hocking	975,481	1843	42	26	4	Carroll, Ohio, to Nelsonville, Ohio.
Lehigh Canal and Nav. Co.	4,455,000	1821	48	57	6	Coalport, Penn., to Easton, Penn.
Louisville and Portland.	5,578,631	1872	2½	2		At Falls of Ohio River, Louisville, Ky
Miami and Erie.	8,062,680	1835	250	97	4	Cincinnati, Ohio, to Toledo, Ohio.
Morris.	6,000,000	1836	103	33	5	Easton, Penn., to Jersey City, N. J.
Muscle Shoals-Elk River Sh.	3,191,726	1890	16	11	5	Big Muscle Shoals, Tenn., to Elk River Shoals, Tennessee.
Ogeechee.	407,818	1840	16	11	6	Savannah River, Ga., to Ogeechee River, Ga.
Ohio.	4,695,204	1835	309	144	4	Cleveland, Ohio, to Portsmouth, Ohio.
Oswego.	5,239,526	1828	38	29	7	Oswego, N. Y., to Syracuse, N. Y.
Pennsylvania.	7,731,750	1839	249	29	7	Columbia, Northumberland, to Wilkesbarre, Huntingdon, Penn.
Portage L. and L. Superior.	528,892	1873	25		15	From Keweenaw Bay to Lake Superior.
Santa Fe.	70,000	1880	10		5	Waldo, Fla., to Melrose, Fla.
Sault Ste. Marie.	4,000,000	1895	3	1	18	Connects Lakes Superior and Huron at St. Mary's River
Schuylkill Navigation Co.	12,461,600	1826	108	71	6½	Mill Creek, Penn., to Philadelphia, Penn.
Sturgeon Bay and L. Mich.	99,661	1881	1¼	..	15	Between Green Bay and Lake Michigan.
St. Mary's Falls.	7,909,667	1896	1¼	1	21	Connects Lakes Superior and Huron at Sault Ste. Marie, Mich
Sus. and Tidewater.	4,931,345	1840	45	32	5½	Columbia, Penn., to Havre de Grace, Md.
Walhonding.	607,269	1843	25	11	4	Rochester, Ohio, to Roscoe, Ohio
Welland.	23,296,323	.	26¾	55	14	Connects Lake Ontario and Lake Erie.

Government Improvements.—Previous to the close of the Civil War the General Government did but little work toward improving the rivers. Some of the States built locks and dams on a few streams and removed obstructions from others. They also granted charters to corporations who undertook the improvement in certain cases. The majority of these works of navigation were, however, allowed to fall into decay after the introduction of competing railroads, and finally they became the property of the General Government, which put them in condition for operation, and in many cases extended the systems. Other streams were also improved until at the present time few rivers

of importance are left uncared for by the Government, and only a few are still in the hands of corporations. Those operated by companies, of course, charge tolls from commerce, but those under control of the Government are free of charge and open to all.

The improvement of rivers by the General Government was begun in a small way in 1827 in the States of Maine, Connecticut, Delaware, and North Carolina, but did not become general until within the last twenty years. No comprehensive plans of improvement have been adopted, but, briefly stated, the operations consist in widening, deepening, and straightening channels by dredging, blasting out rocks in shoals and bends, removing snags, trees, wrecks, and other obstructions with boats and appliances built for the purpose; constructing wing-dams, training-walls, jetties, and dikes to contract channels when necessary to maintain the requisite depth or to give direction to the currents; erecting piers or cribwork for the protection of craft from ice and storms; grading and protecting caving banks, making cut-offs at bends, and canals around falls or rapids; constructing levees to prevent overflow; and the building of locks and dams to furnish slackwater navigation.

Administration.—In the United States, as in most other countries, the navigable rivers are under the control of the General Government. The waterways are a part of the public domain, administered in the interests of the general public, and constitute an excellent regulator for the transportation rates of the country. They are open to all, with liberal regulations for their use. The improvement works are not only constructed but they are also maintained at the public expense, and the more important streams are lighted at night in order to render navigation safe.

The preparation of streams for navigation, as well as their maintenance, are administered under the direction of the War Department by the Corps of Engineers, while matters pertaining to navigation itself, such as lighting, regulations for navigators, etc., are directed by the Secretary of the Treasury. This work of river and harbor improvement is under the general direction of the Chief of Engineers, whose headquarters are at the National Capital, with a staff of immediate assistants. For the better execution of the improvements the country is separated into divisions, and these again into districts, over each of which an engineer officer has charge. Each district officer is assisted when necessary by other officers and by civilian engineers, superintendents, inspectors, overseers, etc., assigned to the special works in the various localities, and is authorized, subject to the approval of the Chief of Engineers, to make and enforce contracts, employ workmen, purchase materials, and make disbursements. Commissions of officers and civilians, and boards of officers, are also appointed when desirable for conducting work or preparing projects. The California Débris Commission, for example, was created to regulate such hydraulic mining as might be deemed injurious to navigation as well as to mature plans for the improvement of rivers which had already been injured by this class of mining.

All expenditures for improvement must first be authorized by Congress, either

by acts appropriating a lump sum for work of a certain character, or a limited sum for specific improvements, or by the authorization of contracts to be entered into for definite amounts, only portions of which as a rule become immediately available, leaving the remainder for future legislation, which, by the first act, is virtually guaranteed. The latter system has in recent years been widely applied, and is known as the "Continuing Contract System."

CHAPTER II.

CHARACTERISTICS OF RIVERS.

Physical Features.—Under similar conditions the same laws apply to all natural watercourses, regardless of size. The smallest creek is characterized by its sinuosities, eddies, bars, caving banks, and overflows exactly as is the greatest river. There is but little definitely known concerning the complicated laws governing these phenomena, but it is of course necessary to take cognizance of this knowledge, scant though it be, in projecting works of amelioration.

The physical features of a country may be resolved into a series of inclined basins or valleys, each drained in its lowest portion by a stream, each stream flowing toward some other watercourse or body of water. Each of these valleys is subdivided into smaller basins, and these again subdivided in the same manner, until the little springs or rivulets at the source are reached. Thus the smaller valleys all lead to a great central valley through which flows a stream of greater or less magnitude, dependent upon the amount of rainfall and the character of the material upon which it is precipitated.

Rainfall.—The cooling of layers of air charged with moisture produces rain or snow, and the presence of forests, mountains, and valleys will also influence the temperature and cause precipitation. The quantity of rainfall in any one locality varies greatly from year to year, and is also much greater at some points than at others, usually diminishing as the distance from the sea or other place of great evaporation is increased. Thus in London, England, the mean annual rainfall is about 23 inches, while in certain districts in India, as shown by the records of the Royal Engineers, the rainfall has reached as much as 600 inches in one year.

Upon reaching the earth the rain-water is either taken up by the soil and vegetation, or runs off into the streams. That portion of the rainfall which runs off and immediately finds its way into the streams causes freshets and floods. Its proportion will vary, like the portion infiltrated, with the character of the material upon which it falls, and the amount of vegetation present. Torelli has stated that a wooded mountain will retain four-fifths of the rainfall, and that the same mountain if denuded will retain but one-fifth, but this is disputed by others. Time is required for the absorption, and this time is furnished where timber covers the ground. Water falling on the foliage is distributed, and upon reaching the earth is held a longer time by the vegetation of various kinds and the fallen leaves. In penetrating the earth it meets conduits through which it is

distributed to the mass beneath. By gravity and capillary attraction, acting in opposite directions, it moves somewhat slowly, but finally emerges from the ground in the form of springs, which are more or less constant as the area of the storage is great or small, and dependent also upon the frequency of renewal by additional rainfall. This portion of the rainfall is very valuable to navigation in that it replenishes the flow of rivers at times when there is no water running off the surface, and thus, in regions containing considerable layers of permeable soils, the summer flow is upheld by the natural storage of the winter and spring rains.

The amount of run-off will also vary considerably with the intensity of the precipitation, much more running off in what is called a hard rain, or thunder-shower, than in a gentle fall. It will also be greater when long continued, the earth then becoming saturated so that it can absorb no more. A frozen soil, or a soil hard-baked by the sun's rays, will turn off much more water than the same soil in its natural condition. The rains of summer, while usually heavy within a short period, are largely evaporated and absorbed by the cultivated earth, but those of winter, falling on soil already saturated and possibly frozen hard, readily find their way into the rivers. If these rains are warm and fall on snow, the conditions are rendered doubly dangerous, as not only the rainfall but also the melting snow will find its way into the already overladen channel. Occasionally, however, the opposite effect has been known to result, and a heavy rainfall has been partially absorbed by a deep snow already on the ground and thus held from running off until the flood had begun to subside. A case of this character recently came under our observation, where a rainfall of 10 inches within 48 hours produced less immediate effect upon the river than half the amount would have done under ordinary conditions. The period of high water was prolonged, but its level was kept down much below that which experience had shown would ordinarily follow such a rainfall.

The melting of snows, when unaccompanied by rains, rarely produces floods in large streams, notwithstanding popular opinion to the contrary. The deepest snows when melted make but a few inches of water, and this reaches the rivers gradually. At night the melting is reduced, or possibly stopped entirely, to be resumed again next day, but even if continuous, with the ground underneath frozen, an unusual condition in many localities, the quantity of run-off will not equal that of ordinary hard rains.

In many parts of the country the rivers are swollen beyond bounds during the spring and early summer months, only to run almost dry in the autumn, and in studying the characteristics of a stream with a view to its improvement, it is therefore necessary to possess information regarding the rainfall, and the data should extend over as long a period as possible.

Bed.—By the term bed is understood all that space ordinarily covered by water and lying between the lands on each side of a stream. In rivers which rise above the levels of these lands and overflow adjacent bottoms, the channel in which the water is usually confined is called the minor bed, while the space occupied during flood-time is known as the major bed. The difference in width between the two is frequently very

great, unless the major bed is confined by embankments. The two sides of the minor bed are the banks, the right bank being always the one on the right hand of an observer looking down stream and the left bank the one on his left hand. In many streams they extend generally above flood-level, while in others the highest floods go over them. In exceptional cases they are submerged by ordinary high-water stages. On nearly all streams there are low banks at intervals, particularly at the mouths of the larger affluents.

The majority of rivers flow through alluvial lands, with varying strata of sand, clay, and gravel as a foundation, underlaid by rock. Here and there the hills approach on both sides, causing a narrowing of the bed, but the presence of hills close upon one side is nearly always accompanied by bottom land upon the opposite side. In such situations, if the soil is easily eroded, the constant action of running water will result in a more or less continual changing of the banks and the bed. The amount of this erosion will vary with the character of the material and the transporting power of the water, which increases with the slope of the stream and with the mass of water in motion.

Basin.—The territory drained by a river is called its basin. It therefore reaches to the mountain-tops and includes the valleys of the tributaries as well as that of the principal stream. Its highest point is the watershed which divides it from another basin, while its lowest point is the bed just described. Its influence on the river is governed by the character of the strata of which it is formed. When these are permeable, the rain sinks into the earth and reaches the bed gradually; when they are impermeable, the water finds its way to the river rapidly, and the river will therefore fluctuate slowly or rapidly as its basin is permeable or impermeable.

Slope.—The slope of a valley determines the velocity of the current of the stream passing through it and is generally greater in the upper portion than in the lower. When it is gentle very little erosion takes place, unless the soil is unusually unstable, but, where the slope is great the water attacks the soil, and the river-bed takes a form dependent upon the quantity of water and its velocity. As the river rises, its slope and velocity will increase, and it is not possible for it in many cases to preserve a fixed bed or banks. From this it follows that the profile cannot remain constant and uniform for all stages and under varied conditions, nor can the cross-section. There is no fixed relation known between slope and discharge, and the former varies with the local conditions of the channel, which also constantly change, so that it may not be the same for any length of time on a given length of river.

Transportation of Sediment.—The mountain-sides and valleys are slowly but surely being carried to the sea by the rivers which penetrate them. In their upper portions, where the slope is great and the velocity sufficient, the material transported is coarse and heavy; as the descent of the valley proceeds this material becomes finer, until near the mouths of rivers it is fine silt, capable of causing much trouble to navigation when deposited in quiet water. This moving material, at whatever point in the river it is

found, leads to most of the difficulties encountered in river navigation. Evidence of it may be seen after every freshet in the deposits left in eddy water, or by allowing a sample of it to settle for a short time. At the bottom of the vessel will be found a small quantity of sediment which may be separated into two general classes, one of very fine particles of mud and clay, and one of sand of varying grades of fineness. The materials of the first class are easily kept in motion, being light, while those of the other grade have a constant tendency toward the bottom, and will be precipitated in places where the current velocity is reduced. Numerous experiments have been made from time to time to determine the velocity at which the current begins or ceases to move various materials, but there are several conditions entering into the problem which render the data obtained of uncertain value.

The experiments of Du Buat give the following results:

Potter's clay	0.26	feet per second.
Sand deposited by clay	0.54	" " "
Large angular sand	0.71	" " "
Gravel, size of peas	0.53	" " "
Gravel, size of beans	1.07	" " "
Round pebbles, as large as thumb	2.13	" " "
Angular flint stone of size of hen's egg	3.20	" " "

Formation.—In order to grasp properly the influence of improvement works it is necessary to understand the natural condition of rivers, and to bring about this understanding we will quote a very clear presentation of the subject by the Russian engineer Janicki: *

"Let us examine how the bends and the bars are formed. To better understand the details of the process, let us suppose a plain, absolutely regular, and not horizontal, but slightly inclined in a certain direction. Let us suppose, further, that in the direction of this inclination a canal is dug having a certain cross-section, a regular bottom, a slope parallel to that of the ground, and regularly constructed side-slopes. Into this canal we will admit a river at its maximum discharge. This river has a certain velocity of current which undermines the banks and cuts out the bottom of the canal, and we shall accordingly witness the following phenomena: the water begins by detaching a few small particles from the bed and cutting away the two banks equally, but as in nature no ground is absolutely homogeneous and of the same tenacity throughout, it finally happens that at some point one bank yields sooner than the other. This first undermining gives rise to a slip or mound which destroys the symmetry of the original profile, and deflects the current toward the opposite shore. Soon this second shore, in its turn, crumbles away just where the deflected current strikes it most powerfully. This new mass of fallen earth cannot remain at the foot of the bank whence it came. The current here being already increased, it is carried a little lower down stream and

* Notes on the Navigability of Rivers. 1879.

there forms a bar, which in turn directs the current toward the bank whence the trouble first started. If we add to this that the current once turned aside from the original straight line wanders farther and farther away by the very force of inertia, and that this deviation continues until, from the very fact of its digression, the slope and velocity of the current are reduced and come into equilibrium with the resistance of the banks, we shall see how bends are slowly and gradually formed, making detours both to right and left of the line of maximum slope. The elongation of bends only ceases when the slope is so diminished from the increased length given to the course through which the water must flow that there is no further tendency to produce scour of bottom or banks. Theory and experience teach us also that velocity of current is determined not only by the inclination of water-surface, but also by the form of the bed; that is, by the form of its cross-section. For any given soil and any given inclination there is but a single form of flowing cross-section which will give the maximum velocity of current with the least resistance. In the above assumed case of an artificial river whose curves are freely developed, the form of cross-section will undoubtedly vary according as we consider the bed at the head of a bend, at its apex, or at the point of passing from one bend to another; these variations, however, will be constant at similar points, for the slope, by reason of the increased length, has become almost uniform, and the other two factors—velocity of current and nature of the bottom—being likewise uniform, the depths of the sections and their widths will have a constant maximum limit. But what will happen if one of the banks is higher or more solid than the other, and if from this, or any other special condition of the surface of the adjacent bottom land, the river cannot sufficiently increase the length of one or more of its bends? It cannot maintain a very steep slope; the nature of the soil forbids that. In such a case it is evident that the stream will more and more scour out its bed, and deposit the débris in those places favored by the topographical features of the valley; that is, where it is possible to build up the bottom. In other words, the result will naturally be an elevation of the bed of the stream, and this elevation will act as a cross-dike, damming the river like a weir; it will withstand and considerably diminish the force of the current; the water will flow over it in a thinner sheet, and, following the exterior slope of the bar, will fall into the lower pool by a route much shorter than through a bend, and without a tendency to cut away its bed; for we know that with a given slope the velocity of bottom flow diminishes with the depth of the sheet of water.

"If the discharge of our artificial river always remained the same, at the end of a certain time it would finally come into equilibrium with the resistance of the soil throughout its whole length. The bends and the bottom, when they had once assumed their proper form, would retain it, whatever might be the consistency of the soil. But the discharge of rivers often varies through wide limits. Into our artificial canal we let loose a river at flood-height. If the discharge should gradually diminish, the water-level would fall. At deep places the section would always be sufficient for the passage of the water in spite of any diminution of the slope caused by lowering the level, and

the regimen of those parts would not vary; but wherever the bottom had previously been raised these elevations would act more and more as if they were dams that closed the whole width of the river. The water would flow over them, would attack them, and dig out a new low-water channel which would have a width and a depth adapted, according to hydraulic principles, to the nature of the obstructions, to the fall (from the upper level to the lower), and to the low-water discharge.

"When the water again rose, the swell, starting at points above, with a current increased with the slope, would bring down new material and fill up the low-water channel already formed, and thus reconstruct the bar to its former height. This work of lowering and reconstructing bars is repeated at each freshet. The longitudinal profile of a river taken during low water shows that it is composed of a succession of pools where the fall is generally less than the mean fall, and also that the pools are separated from each other by bars where the fall is greater than the mean fall of the river.

"We have taken a purely theoretical river for an example, and we have supposed that in the beginning it had a regular bed situated in a plain where the soil was homogeneous. If we now take into account the diverse topographical features, and the geological complexity which ordinary river valleys present, we shall have that variety of cross-sections, of surface slopes, and of more or less pronounced bends which are found in free rivers.

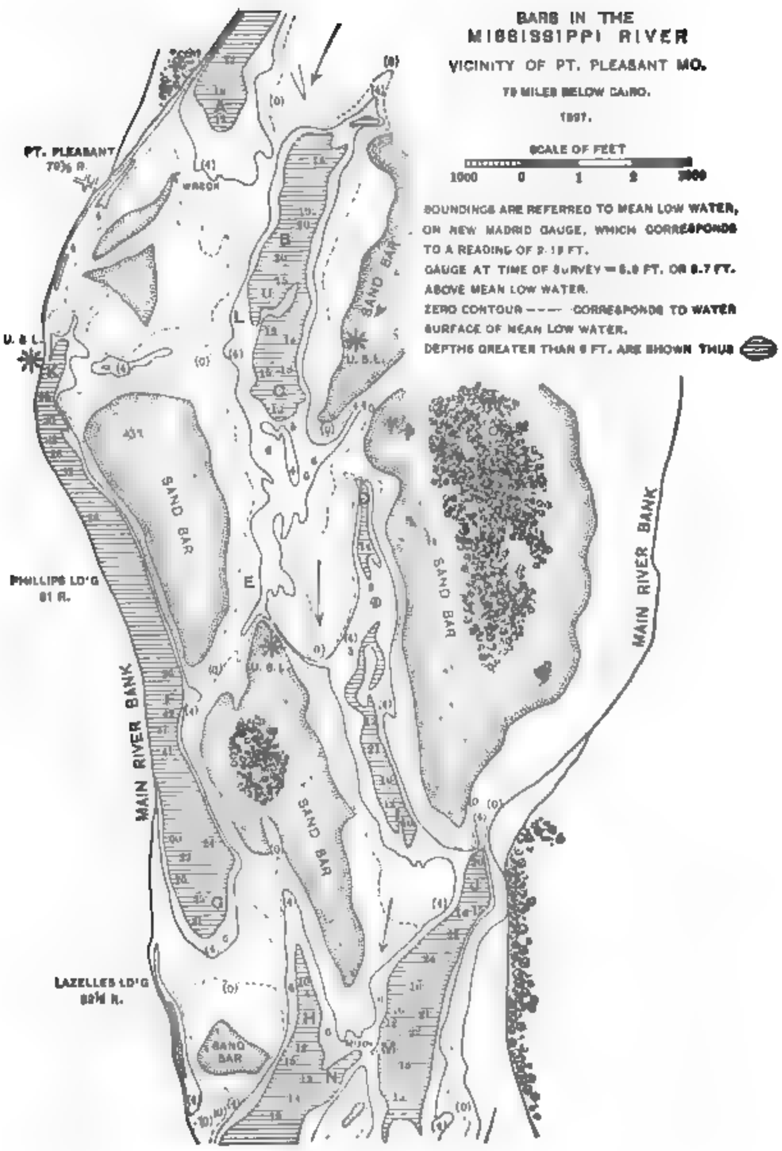
"Our attention is thus called to the intimate relationship that exists between all those phenomena which at first view appear so entirely distinct from each other. No one can anywhere interfere with the curvature of a river, its slope, or the depth of its cross-section, without immediately causing, either above or below the point, some change in the pre-existing conditions of its equilibrium. This equilibrium, we must not forget, is not a static, but a dynamic equilibrium, and, therefore, in the present condition of the science it is very difficult to determine its exact conditions in advance.

"We have shown that the want of solidity in the soil is the natural regulator of the rapidity of the current. This lack of solidity, consequently, leads to the formation of bends and bars, which re-establish the equilibrium between bed resistance and velocity of current.

"The status, the general character of a river, therefore, depends on the united action of these three factors: the discharge, which is variable; the magnitude of the slope, which is likewise variable; and the nature of the soil, which is variable in different localities.

"For a river to be navigable it is necessary that it should have a sufficiently deep channel throughout its entire length. A river may have much water, but if the fall is considerable and the soil unstable it cannot have a deep channel. On the other hand, there are rivers with a relatively small discharge and great fall which are yet quite suitable for navigation owing to their hard bottom."

Shoals.—Shoals, as generally understood, result from a natural disposition of the river-bed which has resisted all erosion, and thus the water is, of necessity, compelled to pass over them. They are, in fact, natural dams of greater or less height and width.


BARS IN THE
MISSISSIPPI RIVER
VICINITY OF PT. PLEASANT MO.
SCALE OF FEET
1000 0 1 2
SOUNDINGS ARE REFERRED TO MEAN LOW WATER,
ON NEW MADRID GAUGE, WHICH CORRESPONDS
ABOVE MEAN LOW WATER.
ZERO CONTOUR CORRESPONDS TO WATER
SURFACE OF MEAN LOW WATER.
PT. PLEASANT
WRECK
U. S. L.
SAND BAR
PHILLIPS LD'G
MAIN RIVER BANK
LAZELLES LD'G

(To face p. 13.)

It is usually not a difficult matter to cut through a shoal and secure the depth required for navigation; but in so doing new troubles are liable to arise because the pool level above may be lowered and new obstructions uncovered. If a cut be made, its limits in width and depth should therefore be rigidly fixed to those actually required, in order to maintain the level above. The reduction of section of flow will also result in increased current velocity which will cause some trouble to ascending navigation and endanger that descending. For this reason the new channel should be as straight as practicable.

Bars.—Bars are formed of material transported and deposited by the water so as to form a ridge, and are usually on the crossings, that is, somewhere in a line which a boat will take in passing from a pool in the bend at one bank to a pool in the bend along the opposite bank. On alluvial rivers of steep slope they are not, like shoals, always at the same place; however, they usually form at about the same point each year. They are the inevitable result of the general travel of alluvium cut from the banks or from the hills near the sources of the stream, and of the accidents of slope common to all river basins, and, while they may be removed from one point, if the material is deposited below high-water line, it is probable that it will be again found in a new bar farther down the river, although it may possibly be in water so deep that navigation will not be disturbed by it. If some of the bars are reduced as fast as formed, it may result in a less number but in greater dimensions of those remaining.

It not unfrequently occurs that bars are formed from external causes, such as the construction of a pier, breakwater, or other structure which may deflect the course of the current in such a manner as to erode the bank or the bed of the stream. In such cases the removal of the bar, in all probability, will not be followed by its renewal if the erosion is stopped by proper protection works. But with bars formed by the transportation and deposit of material on its journey from the mountains toward the sea, no removal by portable appliances or other means can prevent their reproduction, and the best that can be hoped for from such sources is a temporary relief.

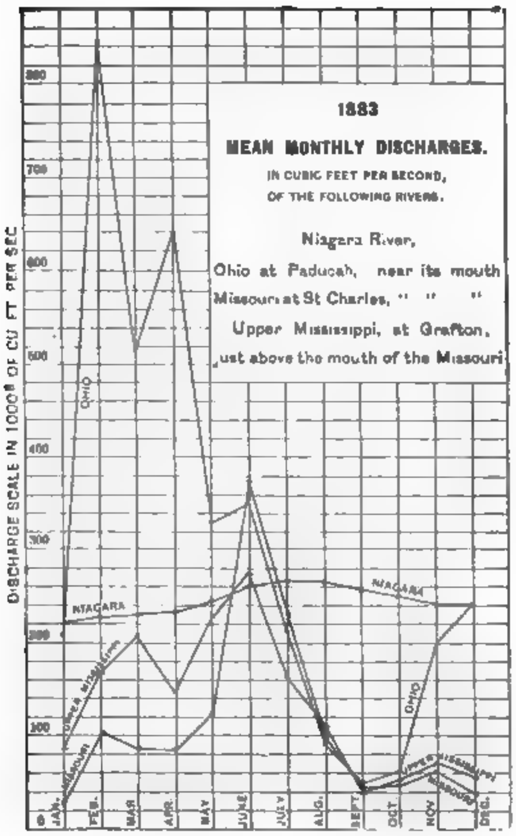
Changes in Level and Floods.—The low-water and high-water levels of a river can usually be easily ascertained, but they are both subject to fluctuation, and the construction of works of improvement may change the conditions to such an extent as to seriously affect previously established levels. On most navigable rivers the lowering of the low-water level would cause a great inconvenience to navigation, and this is true also of the filling up of the river-bed. As a general thing the raising of the high-water level will not seriously affect navigation, but it may cause loss and suffering along the adjacent lands, and is, therefore, to be avoided. As a country is cleared up and cultivated, and the forests removed, the water carries vast quantities of material into the stream, because much of the rainfall reaches the river rapidly under the changed conditions. The result is that floods are more frequent and more violent, and the banks of the river and its tributaries are cut away and carried into the stream. This action is aggravated by the employment of splash-dams and the removal of obstructions in

the smaller streams for the purpose of flooding out the timber therein. This constant inflow of material under the influence of great velocities and steep slopes has the result of gradually filling up the pools and reducing the depths for navigation. In one instance which has come under our observation surveys made of a river of steep slope in 1875 showed a series of deep pools separated by shoals. After that time the country was stripped of much of its timber, farms were opened up, splash-dams were built in many of the tributaries for bringing out logs, the river itself was improved by clearing its bed and confining its channels, and to-day such a thing as a deep pool is not known. They have practically all filled up, so that where, twenty years since, the water was 20 feet deep at low stage, it is now not that many inches. The pools have not only filled up but the low-water level and river-bed have been raised in the lower portion of the stream, the tendency being to form a bed with a regular slope, and in many cases these grade-lines have obliterated the smaller shoals entirely. This new bed is in many places entirely above the old low-water level. Thus in the lower part of the stream a gauge was set with its zero 9 inches below low-water mark. Ten years later the low-water mark was 2 feet above the zero, with really a less discharge in the river than at the time the gauge was fixed, and in the same period all the shoals and pools within 10 or 15 miles of the gauge had disappeared and regular slopes had been established. This action was not due to the construction of any improvement works in the vicinity.

In the study of a project for open navigation it is of importance to allow for the variation between the approximate low water which is known and the extreme low water which may occur under certain conditions, but the position of which is not known. The approximate stage is generally called conventional low water, and is a plane of comparison a little arbitrary, but almost coinciding with the lowest water, and consequently giving a basis on which to work. The next stage above conventional low water is ordinary low water, that is, the depth ordinarily reached by the river at periods when there is barely sufficient water for navigation without a reduction of loads. At this stage boats proceed with more care and less speed.

The highest navigable stage is also one of importance, particularly in fixing the level of bridge floors. It is seldom practicable to raise the coping of works of navigation above the level of the highest navigable water in this country, although this practice obtains in parts of Europe where the flood variation is not so great. The level at which navigation ceases necessarily varies, and can be increased if boats are built stronger, and if more powerful means of propulsion are employed. When a river reaches a certain stage, however, navigation becomes very difficult, and even dangerous, on account of the increased velocity of the current as well as from the uncertainty of the directions of its forces. Ordinarily navigation ceases when the water overflows the banks, but it will vary with circumstances and at different points on a stream according as the banks are high or low, or the river comparatively straight or crooked.

It is also of importance to know the height of the greatest flood. Like conven-


1883
MEAN MONTHLY DISCHARGES.
IN CUBIC FEET PER SECOND,
OF THE FOLLOWING RIVERS.
Niagara River,
Ohio at Paducah, near its mouth
Missouri at St Charles, " " "
Upper Mississippi, at Grafton,
just above the mouth of the Missouri
DISCHARGE SCALE IN 1000S OF CU FT PER SEC
700
600
500
400
300
200
100
0
JAN.
FEB.
MAR.
APR.
MAY
JUNE
JULY
AUG.
SEPT.
OCT.
NOV.
DEC.
OHIO
NIAGARA
UPPER MISSISSIPPI
MISSOURI

(To face p. 15.)

tional low water this level may change in the future, and these changes depend upon conditions which cannot always be ascertained or guarded against.

American rivers are subject to floods of much greater height than are those of Europe, where floods on large streams rarely exceed 50 feet above ordinary low water. The Ohio River has had a number of floods exceeding 50 feet at Cincinnati and that of 1884 was over 70 feet.

Where rivers are fed by lakes they are not subject to such great extremes of level. This is particularly noticeable in the river St. Lawrence, which has an approximately constant level. To a certain extent the same is true of the Rhone, the Rhine, and a number of other streams.

as the depth and width of the proposed channel, is of course influenced by the nature and the amount of commerce, present and prospective, which should be commensurate with the cost. Other considerations would be the dimensions and character of existing works of navigation on other parts of the stream or on its tributaries, or on the stream to which it is a tributary, the supply of water, the nature of the bed and banks, the slope, and action of floods, etc. Unless the survey has been exhaustive only an approximate estimate of cost can be arrived at. This should give in detail, as far as practicable, the quantity and cost of the various items forming the several parts of the improvement, and may also state the probable cost of maintenance. If the improvement is to be by slackwater, then the number of locks and dams with the dimensions and general characteristics, as well as probable cost, should be given. Approximate locations may be fixed for the purpose of making the estimate, but the final locations are not generally decided until the time has arrived to prepare for construction, as changes in the river from time to time and the effect of other dams already built should be ascertained first, and more careful borings and investigations may point out a better location than would be indicated by the first survey.

In determining upon what part of a river to begin the improvement, whether at the lower or upper end, or at some intermediate point, the requirements of present and prospective commerce should be considered, and those places forming the worst obstructions should be first overcome.

Preparation for the Improvement.—After a project has been approved and adopted, and sufficient funds appropriated for carrying it out or for commencing it, the engineer officer in charge of the district usually assigns an assistant engineer to take local charge of the improvement, although sometimes another engineer officer, acting under the immediate orders of the district officer, is placed in local charge. If the work be of minor importance, or if it be simply the removal of obstructions to navigation, it is frequently assigned to a master workman or to a steamboat- or river-man familiar with the requirements of the locality. This character of work and the building of small dikes, wing dams, etc., in the smaller streams is difficult to let by contract, and the usual method is to do it by hired labor, working under an overseer. It may require the use of boats, derricks, explosives, and a variety of appliances and tools which the Government will hire, purchase, or construct, as may appear best in each case.

Plans and Specifications.—When the location has been fully decided upon and the various dimensions determined, plans and specifications are prepared, if the work is one of sufficient magnitude to require them. These will comprise complete plans, sections, and elevations of the work to be built, as well as detailed drawings of special parts, and a full description of the work to be done and manner in which it is to be executed.

When there are insufficient funds to complete the work, or for other reasons, it may be advisable to make a contract for the preparation and delivery of the necessary material, depending on future appropriations for money with which to carry on the building. Later contracts may be made for the construction, or different contracts

may be made for the several parts of the work. However, it may be advisable to make no further contracts after the delivery of the materials, but to build by hired labor. This method is frequently adopted where funds are insufficient to enable an economical contract to be arranged, but it has the objection that it requires the Government to purchase or hire a plant, and it also requires, if it is to be economically carried out, an engineer with a good practical knowledge of construction in all its details, such as the use of engines, boilers, derricks, and appliances of all kinds; in fact, he should be not only a good civil engineer, but also a practical master workman.

The specifications for furnishing materials should describe the character and location of the work and the facilities for transportation, and should state what the contract is to include, the quality and quantity of materials required, fully classified and each class described; the dimensions, the style of finish of each class, when and in what proportions of each class delivery is to be made; the time the contract will expire, etc., etc.

The specifications for building should state what the contract is to include, describe the nature and depth of foundations, kind of coffer-dam required, amount and character of excavation and how classified, classification of masonry and method of ascertaining quantity, manner of placing cut stone, timber, concrete, filling, etc., quality of cement, proportions of mortar and method of mixing, manner of fastening irons and anchorages, etc., etc.

Both drawings and specifications should indicate that the best class of material and construction will be required, as long experience has shown that where a work of improvement is intended to be permanent, as in slackwatering a river, the truest economy lies in a construction that will require a minimum of repairs.

CHAPTER IV.

TOPOGRAPHICAL SURVEYS AND LEVELING.

TOPOGRAPHICAL SURVEYS.

A TOPOGRAPHICAL survey of a river is made for the purpose of locating its course geographically, and to establish such points as may be required in the prosecution of the works of improvement. The survey may be made with (1) the transit and chain, or tape, and level, as is customary in railroad work, or (2) with plane-table and stadia-rods, or (3) with transit and stadia-rods, or (4) by triangulation.

(1) The methods of **Transit and Chain** surveys are so well known as to call for but little explanation. If the stage of the river is sufficiently low, it may be possible to make the survey in the bed, and this will greatly simplify matters, because an approximately level surface will be secured for the measurements, without obstructions from brush, buildings, timber, etc. When this condition does not exist, it will be necessary to run a line along one or both banks. Stakes should be set every hundred feet, and from these offsets may be made to determine the shore lines and other points desired, such as high- and low-water marks, rocks, bars, etc. The angles are of course measured with the transit, by which the stakes are also lined in, and the observations should be approximately checked with the magnetic needle. The level party follows the transit, taking levels to the water surface at sufficiently close intervals to determine the slope, and establishing bench-marks at points from half a mile to a mile apart, or checking upon those of a former survey, if any has been made. The topographer closely follows the level party with a hand-level and note-book, recording the general cross-section of the territory passed over, either pacing his distances, or, where great accuracy is desired, using a tape-line. He should also make sketches, more or less complete, of the general features of the line, as these are often of much value in plotting the survey. His notes also give the contours of the ground at certain intervals, say 5 feet.*

A transverse profile or section of the river-bed should be made at or near each bench-mark, and at the probable locations of works of improvement such sections should be taken from 100 to 400 feet apart, by one or other of the methods described in the next chapter. Notes as to the character of the bottom and of the banks should also be made, so that an approximate geological section can be constructed.

* A more complete description of the methods of topographical and hydrographical surveying will be found in "The Theory and Practice of Surveying," by Professor J. B. Johnson.

(2) **Plane-table** surveying has had an extensive application upon river and lake surveys in the United States. A plane-table is, as ordinarily used, a drawing-board, provided with a movable straight-edge or "alidade," to which a telescope is attached. The table is mounted for convenient use upon a tripod, and is provided with bubbles for leveling and a needle for magnetic bearings. As the rule moves exactly as does the telescope, it is possible to lay out at once upon the paper fastened upon the board all the directions taken by the telescope to whatever scale is desired.

To use the plane-table, it is first necessary to establish a base line of known length and to plot it upon the paper to the scale proposed for the map. The instrument is then set on one extremity or station of this line, and the fiducial edge of the ruler is brought into coincidence with its two points, and the table revolved until the opposite station is in the line of sight. By clamping the table and using the slow-motion screw the setting is completed. The directions of any other objects or points may then be plotted by moving the alidade, short lines being drawn in at their supposed distances, and marked in such a manner as will identify them. When all the directions have been taken that are desired, the table is set up over the other station of the base line, from which directions to the same objects are taken and plotted. The intersections of these new lines with those taken from the first station will accurately fix the positions of the objects themselves upon the paper.

For points of great importance it is well to take observations from still another station, and thus have a check upon the accuracy of the work.

The topography is added to the sheet as the work progresses, either by the use of stadia-rods or tape or chain measurements.

The level must of course be used, as in the method first described, for obtaining the profile and elevations, and fixing the bench-marks.

(3) **Stadia** surveys are frequently sufficiently accurate for river work, and this method, which is rapid and economical, is applicable to rivers up to one-half mile in width. By its use it is possible to dispense with the chain, and sometimes with the level also, if only approximate results are required. Measurements are obtained with as much accuracy as can be done with a chain in ordinarily broken country.

In the stadia method the distances are measured by noting what portion of a graduated rod is seen between two crosswires placed in the telescope of the transit for that purpose. With most instruments these wires are so placed that a division of 1 foot on the rod will exactly coincide with the space between them at a distance of 100 feet from the instrument, plus a certain amount (varying with each instrument, and usually between 11 and 15 inches), dependent on the focal length of the telescope. For levels as accurate as are required for the location of locks and dams it is necessary to use the level, as in the preceding methods, but for ordinary elevations the vertical circle will give the angle of an object, and the tangent of this angle into the distance measured horizontally will give the distance below or above the instrument.

Where the river to be surveyed is wide, it will be necessary to have two transit

and level parties, and one hydrographic party, each being provided with the necessary skiffs, etc. A line is run along each bank and marked with stakes and bench-marks in the usual way, connecting angles being turned every two or three miles on stations on the line on the opposite bank, for the purpose of checking. The two levelers should make occasional comparisons in the same way. All directions should be compared with the magnetic needle as the work progresses, although in a hilly country, and especially where beds of iron ore exist, the check is only a rough one, as the needle is easily influenced by local attraction. Small isolated hills in a level country will also deflect it. As in a chain survey each party must be provided with a topographer. The average error of closure should not much exceed 1 in 600.

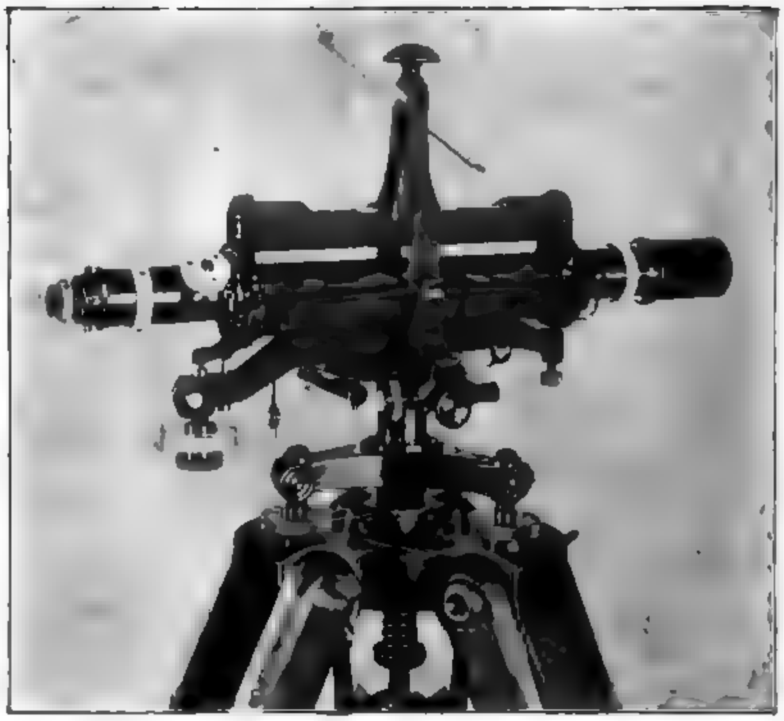
The hydrographic party also has a transit-man, who works from points on the shore from ½ to 1 mile apart, and connected by the other transit-men with the general survey. His duty is to locate the position of the soundings by angles and stadia-distances, a rod being held in the skiff for that purpose. The recorder in the skiff chooses his ranges by eye, keeping as near as practicable in straight lines, and both transit-man and recorder keep tally on the soundings by a time-check, supplemented by special signals at every tenth or twentieth one, as may be desired.

On large rivers the rate of progress of the parties may average 1 to 1½ miles a day, while on small rivers the rate may reach as high as 6 miles.

The accuracy of stadia surveys depends largely on the clearness of the atmosphere. In the U. S. Lake Surveys the average length of sight was 800 to 1000 feet, with a maximum of 2000 feet. The lines averaged 1½ miles in length, and a limit of error in closure was allowed of 1 in 300.

(4). **Triangulation** furnishes the most complete system of surveying, because by it latitudes and longitudes and the location of all salient features may be determined with precision. It is applicable to rivers of great width where other methods would not be satisfactory or possible, as well as to those of smaller size. Briefly described, it consists in dividing the country to be surveyed into a series of large triangles, one side of one of them being exactly measured for a base-line, and all the other sides and angles being found by triangulation. One side of the last triangle is also measured, thus serving as a check on the whole work. These large or primary triangles are subdivided into smaller or secondary triangles, and these again into tertiary triangles, thus embracing the whole district. The details are filled in by topography. In this system no angle should be less than 30°, as a flat intersection is conducive to error. In the survey of the Mississippi River * the maximum closing error allowed in the triangles was 6 seconds. This required the greatest care in the observations, as an error of one-third of an inch in centering a transit will make a difference of one second in the angle in a distance of 1 mile. The base-lines in this survey were placed about 75 miles apart, and were measured two or three times, the allowed discrepancy being 1 in 250,000. A 300-foot steel tape was used, supported every 30 feet by hooks fastened to stakes, and stretched to

* Annual Report, Chief of Engineers, U. S. A., 1891.



BUFF AND BERGER PRECISE LEVEL, NO. 2768.

(To face p. 23.)

a uniform tension by a weight of 16 pounds, with a thermometer every 100 feet. The station distances were marked on strips of zinc fastened to the tops of the stakes. The position of each base was checked by astronomical observation. In closing on triangulation points the error allowed did not exceed 5 minutes for the angles and 1 in 500 for distance. Contours in the bottoms were taken 5 feet apart, and 20 feet apart on the bluffs, and stadia sights were limited to 500 meters.

Permanent survey marks were placed about every 3 miles, two to a set, and on each side of the river on a line normal to the stream, one being placed near the bank, the other half a mile back. They were made of iron pipes set on a flat tile, and with the tops projecting about a foot above the ground.

LEVELING.

Levels may be divided into Ordinary and Precise.

Ordinary levels are run with a regular wye-level, and the degree of accuracy attained depends principally upon the care with which the work is done. Good leveling, whatever be its kind, requires a steady atmosphere, with sights of equal length, not over 100 meters under favorable conditions, and reduced to one-third of that when the atmosphere is unsteady. The level should always be kept in the shade as far as possible, and should be tested daily for adjustment. Both the level and the rod must be solidly set, and care taken always to see that the bubble is in the proper position at time of reading. It is also necessary that the level-rod be held exactly plumb.

In the level work of the U. S. Geological Survey the sights were limited to 300 feet, and were made equal as nearly as possible, the distances being obtained by pacing. Work was suspended during high winds or when the atmosphere was "boiling" on a hot day. The rodmen, who were required to use plumbing-levels on the rods, were supplied with conical steel pegs, 6 to 12 inches long, which were driven into the ground for turning-points, while for bench-marks copper or bronze bolts, set in rock or iron posts, were used. The error of altitude was limited to $0.05\sqrt{\text{distance in miles}}$.

On the Mississippi River survey, where a line of levels was run along each bank, the parties were required to check on each other every 3 miles, and the error of closure was limited to 0.2 foot for ordinary lines, and to 0.05 foot for the bench-marks, which were placed about a mile apart.

Precise levels are made with an instrument constructed for the especial purpose. Their object is to determine with accuracy the elevations, with reference to sea-level, of inland points too distant for reliance upon ordinary methods of leveling. Most countries have therefore adopted the system of precise leveling for ascertaining the elevation required, and the United States has been using it for a number of years in its more complete surveys, establishing permanent bench-marks at short intervals.

The accuracy with which this class of work is done is not all due to the instrument used, but much of it must be credited to the care taken and the methods employed.

CHAPTER V.

HYDROGRAPHIC SURVEYS.

HYDROGRAPHIC SURVEYING comprises that part of river surveying which has to do with the water itself. It may be treated under the following heads:

(1) Soundings.
(2) Discharge.
(3) Slope.

These comprise such matters as depth, velocity, sectional area, material held in suspension, location of obstructions, etc., and such surveys are generally made in connection with a topographical survey of the banks. It is, however, necessary on some large streams with shifting channels to make frequent hydrographic surveys in certain localities purely for navigation purposes.

(1) SOUNDINGS.

Cross-section.—The discharge which a stream has at a certain height of water is usually found by taking a cross-section at right angles to the direction of the current and multiplying the submerged surface or wetted perimeter of this profile by the mean velocity of the current, according to formula. The section should be taken at a point where the river is of uniform width and of regular form for some distance above and below, so that the directions of the currents may be practically parallel, thus avoiding oblique velocities. As sections of a river-bed are quite irregular and would not be the same at points but little separated, it is usually best to take several sections at established intervals, and thus secure a mean cross-section. The stage of water at the commencement of the work, together with changes which take place during its progress, should be known and recorded.

When the soundings have been completed, and levels taken for that portion of the river-bed which lies above the water at the time of sounding, the whole can be plotted and the area determined for any stage proposed. The points at which soundings are taken along the range-line should be accurately located, so that each day's notes may be referred to the same ground.

Methods.—By a Cord Across the Stream.—Where the river is not too wide, a strong cord, which has been well stretched and wet for some time, or a wire, may be stretched from shore to shore. The latter is much to be preferred, however, as a cord will stretch

and contract daily. Desired distances are marked by tags made of metal or of leather, and attached by light wires or twine to the cord or wire. The depth of the water at these tags may then be found by the use of a graduated pole or a sounding-lead manipulated from a rowboat. The pole should be heavy at the bottom, with a flat end, so as to give the depth accurately without sinking into the material of the bed. In deep water and swift currents it will be necessary to resort to the lead. This is a heavy, slender weight, attached to a chain or rope which has been stretched and graduated. If a rope is used, tests of the line should be made often and its variations noted, in order that proper corrections can be made in reducing the notes. The usual method of obtaining the depths is to permit the sounding-boat to float down stream and let the lead strike bottom just as the range-line is crossed. If a steamboat is employed, the line is stretched along the guards with the lead at the bow, from which point it is dropped into the water and the reading is taken at the stern when the line becomes perpendicular.

Another method is to row the boat across the river along the range-line. The leadsman occupies the bow of the boat which is kept well up under the wire, and near him is the recorder, who makes a note of the stations and the depths called out by the leadsman.

By Range-line and Instrument.—In wide streams a range-line is established and defined by two numbered targets on the shore, and the sounding or depth stations are fixed by angles taken by a transit placed on a base-line on the bank, or by a sextant on the boat. The details of the method are as follows: *

"For cross-section soundings all taken are 'drifting soundings.' The boat is run up above the cross-section line being sounded, the lead cast and held about one foot from the bottom. As the boat drifts past the range-line, the lead is allowed to touch the bottom, the depth noted and recorded.

"In shallow water the soundings are taken from a skiff, a steam-launch being employed in the deep water and rapid currents.

"Each cross-section is marked by two range-signals. These are made either of one-inch boards about three feet square, painted white, or of white cotton, fastened to a frame of about the same size. These are attached to a post about 12 feet in length, securely planted in the ground, or to a small tree, the front signal being placed near the river-bank, and the rear signal from 500 to 1500 feet inshore. This arrangement clearly defines the range-line at any point in the river, and enables the leadsmen to determine when the boat is exactly on range, so that soundings can be obtained on line, and renders unnecessary the use of more than one instrument for locating the soundings.

"The location of all soundings taken is determined instrumentally as follows:

"Those taken from the steam-launch are located by an observer on board with a sextant, by one angle to two of the stations on shore. When soundings are taken from a skiff, an observer with a transit occupies one of the stations on shore, at a sufficient

* Annual Report, Chief of Engineers, U. S. A., 1883, pp. 2192 and 2210.

distance above or below the range that is being sounded to give good intersections. The skiff is rowed a sufficient distance above the line being sounded to allow the lead being cast and held near the bottom, and the boat to drift along with the current, so that as the range is crossed the lead-line is as nearly vertical as possible; at the moment of crossing the range the recorder calls 'Sound!' and raises to a vertical position a small flag which he holds in his hand as a signal to the observer on shore. The position of the lead-line at the time of sounding is located by an angle taken from shore. The depth of water and the nature of the bottom is entered by the recorder, and the boat is rowed to a new position. The cross-section soundings thus obtained are generally from 50 to 100 feet apart.

"The lead used is about 22 pounds in weight, slightly hollowed out at the bottom, the space being filled with tallow so as to obtain specimens of the bottom at every cast of the lead. The line is an ordinary ½- or ⅜-inch cotton line, or one of Italian hemp or sea-grass, well stretched and marked with leather tags at every foot; it must be frequently tested. These tests should be made from day to day, and when the length of the line has changed a sufficient amount to affect the accuracy of the work the line must be retagged. A swivel attached to the line near the lead will prevent the line from twisting when in use.

"Longitudinal soundings are taken from a skiff. The range across the river is divided up into about equal distances so as to allow ten lines to be taken in the width of the river. The boat is located in position by the aid of range-signals on shore. An auxiliary range is placed about 100 feet above the upper range, so that the boat can be rowed to the intersection of the two ranges, the lead-line run out, and everything be in readiness to begin sounding as the boat drifts past the upper range. By this time the boat has lost its momentum, due to rowing, and is drifting along naturally with the current.

"**By Two Transits.**—For cross-section soundings the position of each may also be determined by intersections from two transits, one at each end of the base corresponding to the ranges to be sounded. The ranges are permanently marked by stakes which have holes bored vertically in their tops to receive movable flags. The range to be sounded is designated by two flags on each side of the river, one being planted in the permanent stake on the levee, and the other in the ground at the water's edge, and carefully lined on the range. These inner flags are always located by the instruments, and their positions recorded as those of the first and last soundings, each with a depth zero.

"Soundings are taken from a launch or from a skiff, the party consisting, in addition to the crew, of a rodman in charge, who acts as a recorder of the soundings, and a leadsman. The boat is signaled up and down stream by a flagman stationed at the shore flag; he directing the boat a sufficient distance above the line, so that it would acquire, by drifting back, such velocity as would prevent the lead-line from sagging. The leadsman is stationed in the bow of the boat, where there is a mast or staff with

a flag so rigged that the steersman can raise or lower it instantaneously. When the boat passes the cross-section line going up stream, this flag is raised and the transit-men bring their instruments to bear upon it, keeping their vertical wires upon the mast as near its intersection with the deck as possible. The boat continues up stream until the flagman on shore signals to sound. It is then allowed to drift astern, the leadsman heaving the lead as soon as the headway is lost and raising or lowering the lead so as to have it on the bottom and to keep the line plumb. An instant before the boat crosses the line going down stream the flagman on shore signals; the recorder calls out 'Sound'; the leadsman gives the reading on his line; the steersman lets the flag drop, and the transit-men read their angles.

"For longitudinal soundings two additional ranges are located at each base, one 1000 feet above, and the other the same distance below the middle range. At the middle of each base oblique ranges are laid off to intersect the uppermost of the new ranges in ten selected positions.

"The positions assigned to the party are the same as in the cross-section surveys, with the exception of the flagman on shore, whose duty consists in changing the flags on the oblique ranges and in signaling the boat when it approaches and when it crosses the middle range.

"Soundings are taken mainly from the skiff, and while it is drifting astern, the velocity being so regulated that the lead-line can be kept plumb. The boat having been run into position, obtained by the steersman from the intersection of the range-signals, is rowed up stream a sufficient distance to obtain a vertical sounding by the time it has drifted back to the upper or zero range. As soon as the boat is in position the flag is raised, and when the uppermost range is crossed going down stream it is dropped. Between ranges the flag is raised at every fifth and dropped at every tenth sounding. As the boat approaches one of the old cross-section lines, the steersman is signaled and raises the flag; when the leadsman is on the line he is again signaled and drops it. At each end of the base this is determined by the observer setting his instrument on line and giving the requisite signals, and at the middle range by the flagman 'lining in.' The paths of the boat are located from the same base as was used in cross-section surveys. The shore flags are placed and located as before, thus giving cross-sections of twelve located soundings.

"**By Range-poles.**—As it may be necessary to measure the cross-section after every rise of considerable extent on rivers having shifting beds, because of changes of form and elevation in the river bottom, it will be well to fix certain range-points by which this work can be done with less instrumental work, and accuracy in finding the same positions secured. Two poles should be placed on each side of the river on the section-line produced. Then points above or below on either or both sides of the river may be selected from which angles will be turned to the section-line and ranges set up on these lines. Thus the observations can thereafter be taken without the use of a transit, all the points having been previously fixed and a diagram made."

(2) DISCHARGE.

By the discharge of a stream is meant the amount of water passing a given point in a given time. It is usually reckoned in cubic feet per second. The purpose of ascertaining the discharge of a stream is to be able to determine its value in the study of the improvements necessary to control it or utilize it for navigation. It varies greatly with local conditions, slope, etc., and reliable results can only be obtained by numerous gaugings covering all the varying conditions of the river, and extending over a considerable period of time.

Observations for ascertaining the discharge of a small river can be made with a fair degree of accuracy, but for large streams with swift currents, irregular cross-sections, and unstable banks, the undertaking is one fraught with unsatisfying estimates to those who would have nothing but precise results. Thus it has never yet been possible to accurately ascertain the extreme high-water discharge of the Mississippi because of the indirectness of its currents, both horizontal and vertical, its numerous changes, and its extraordinary width. Attempts have been made to compute the discharge by means of percentages of rainfall, but the results are not reliable because of the uncertainty as to the percentage which reaches the river.

Discharge over Weir.—A satisfactory manner of gauging a stream, with results probably within five per cent of the truth, consists in concentrating all the water so it will flow over a weir. It is easy then to get the length and depth of overflow and thereby arrive at the discharge. Weirs for this purpose should be placed at points of impermeable bottom in order that no leakage or percolation may take place: otherwise the discharge measured will be less than the actual volume.

Free Discharge.—The latest experiments for determining discharges by this method were made in 1899 at Cornell University, New York State, by Mr. Geo. W. Rafter and Professor Gardner S. Williams,* and are the only ones up to the present time in which a considerable depth of water was used on the crest. In the experiments of Bazin, made a few years previously, a maximum depth of about 18 inches was allowed, while in those at Cornell University the depth in some of the experiments was over 4½ feet. From the latter the accompanying diagram of discharge curves was prepared, and summarizes conveniently the results of the investigations, combining them at the same time with the formula deduced by Bazin. This formula, which is for free discharge, that is, where there is no water below to check the flow over the weir and no contractions at its ends, and neglecting the velocity of approach, is as follows:

$$Q = mlh\sqrt{2gh}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

* Proceedings of the American Society of Civil Engineers, March, May, August, 1900. The accompanying diagrams are reproduced from the May "Proceedings," from the discussion by Professor Williams.

where the coefficient $m = \left\{ 0.405 + \frac{0.00984}{h} \right\} \left\{ 1 + 0.55 \left(\frac{h}{p+h} \right)^2 \right\}$

and Q = discharge in cubic feet per second;

l = length of weir in feet;

h = height of water above crest in feet;

p = height of crest in feet above bottom of channel of approach;

g = acceleration of gravity = 32.2.

The height h is the head or distance from the top of the crest to the normal surface of the approaching water, or the level at which the water would stand if the velocity of fall over the crest had not reduced it. In Bazin's experiments it was measured at 16.4 feet, and in those at Cornell at 38 feet above the weir.

The profile of the falling water is shown on the accompanying diagram platted from measurements made during the work.

Where h is between 4 inches and 1 foot Bazin gives $m = 0.425$ approximately. Trautwine gives the following table of values of m:*

Head H, Feet.	Sharp Edge. $m=$	Two Inches Wide. $m=$	Three Feet Wide; smooth, sloping outward and downward from 1 in 12 to 1 in 18 $m=$	Three Feet Wide; smooth and level. $m=$
0.08	.41	.37	.32	.27
0.25	.40	.39	.34	.31
0.50	.39	.41	.35	.33
0.66	.39	.41	.34	.31
0.83	.38	.40	.34	.31
1.00	.38	.40	.33	.31
2.00	.37	.39	.32	.30
3.00	.37	.39	.32	.30

Submerged Discharge.—Where the weir is submerged, that is, where there is water below to check the discharge, the formula generally employed assumes that the water above the lower pool level discharges into free air and that that below discharges through a submerged opening. It is as follows:

$$Q = cl(h + \tfrac{2}{3}d)\sqrt{2gd}; \qquad (2)$$

where Q = dischaige in cubic feet per second;

c = coefficient of discharge $= \frac{\text{actual discharge}}{\text{theoretical discharge}}$;

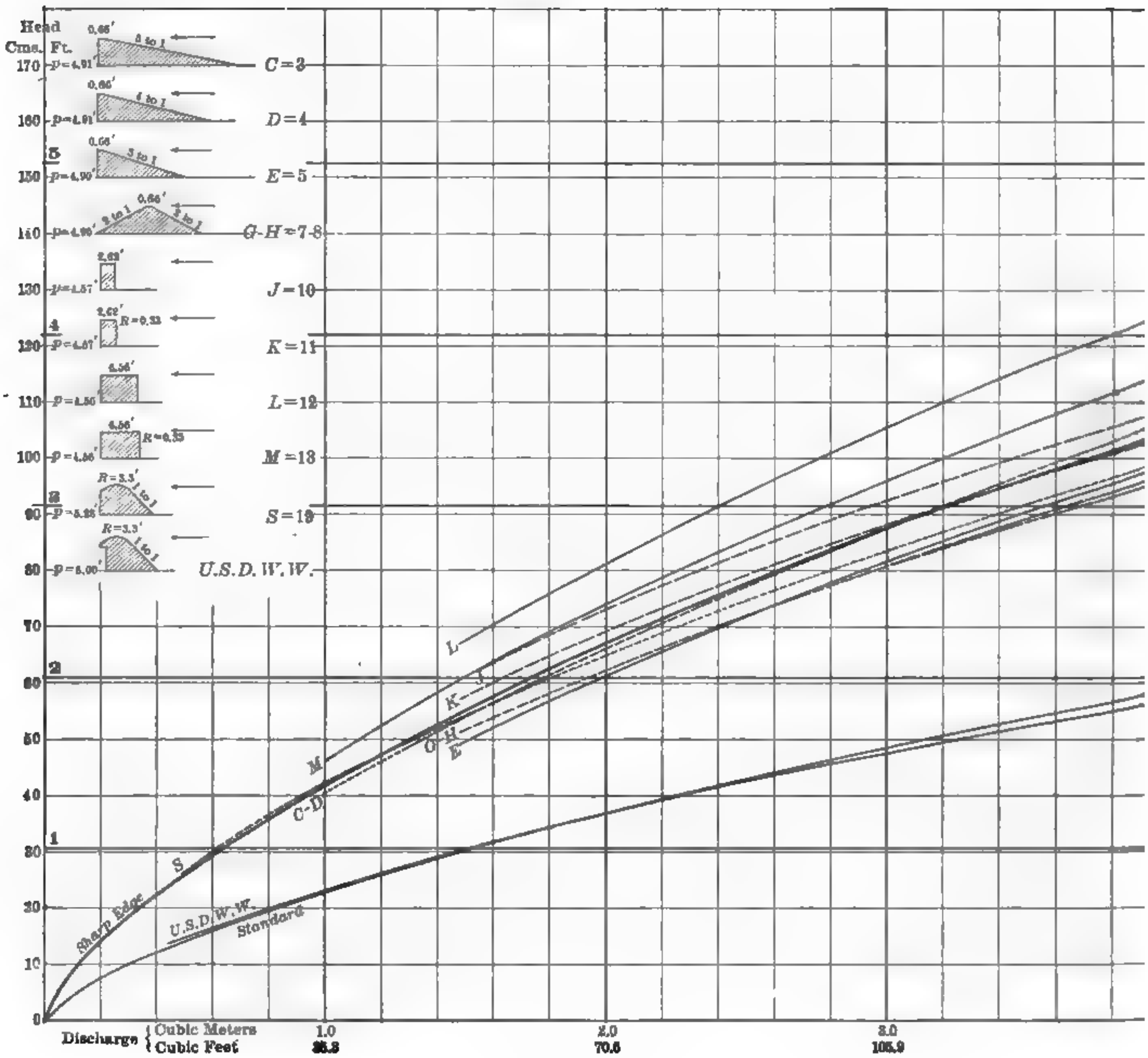
l = length of weir in feet;

h = height of lower pool in feet above crest of weir;

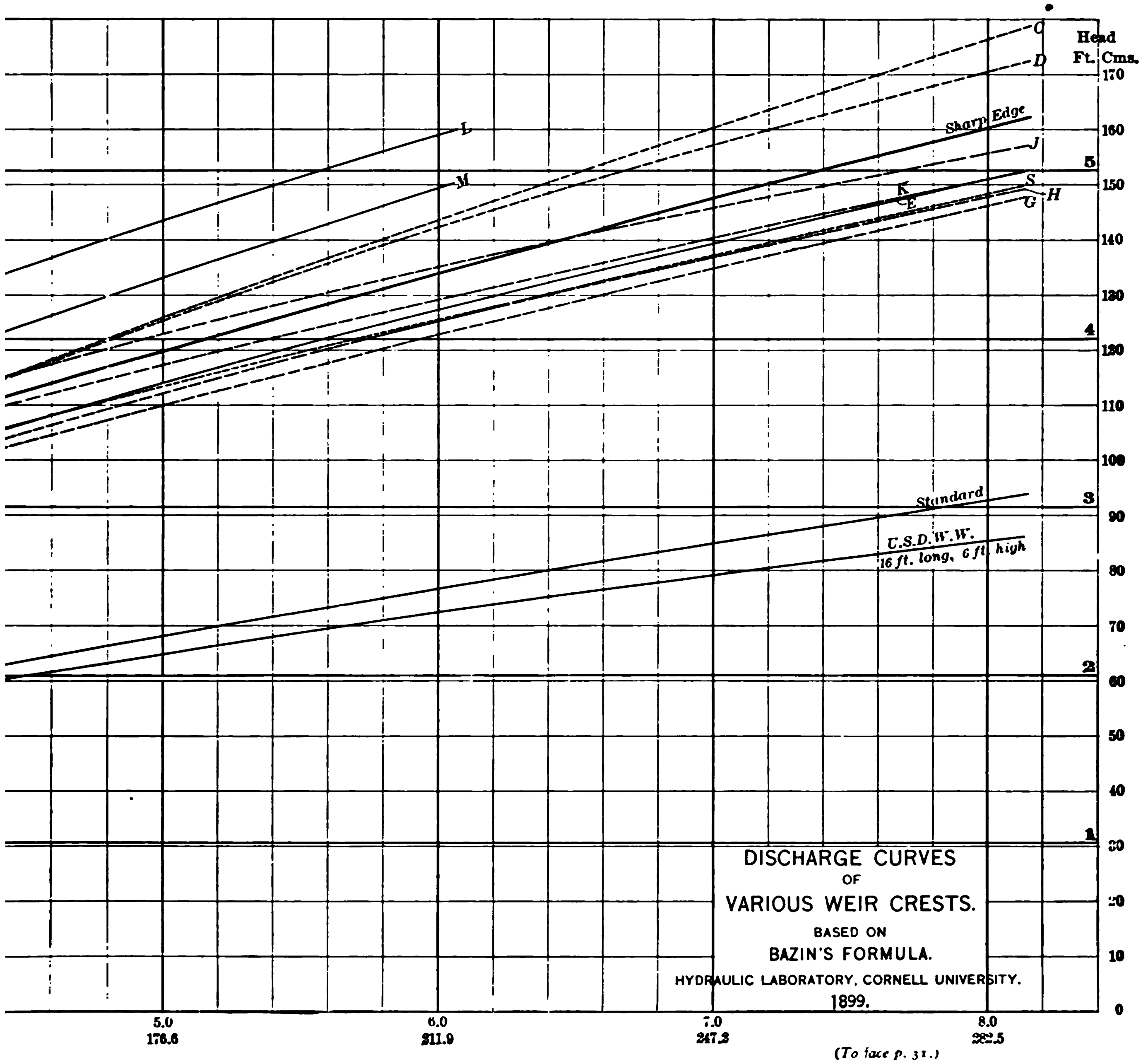
d = difference between upper and lower pools, measured to still water, as in Bazin's experiments;

g = 32.2 feet.

* Engineer's Pocket-book, p. 267e.

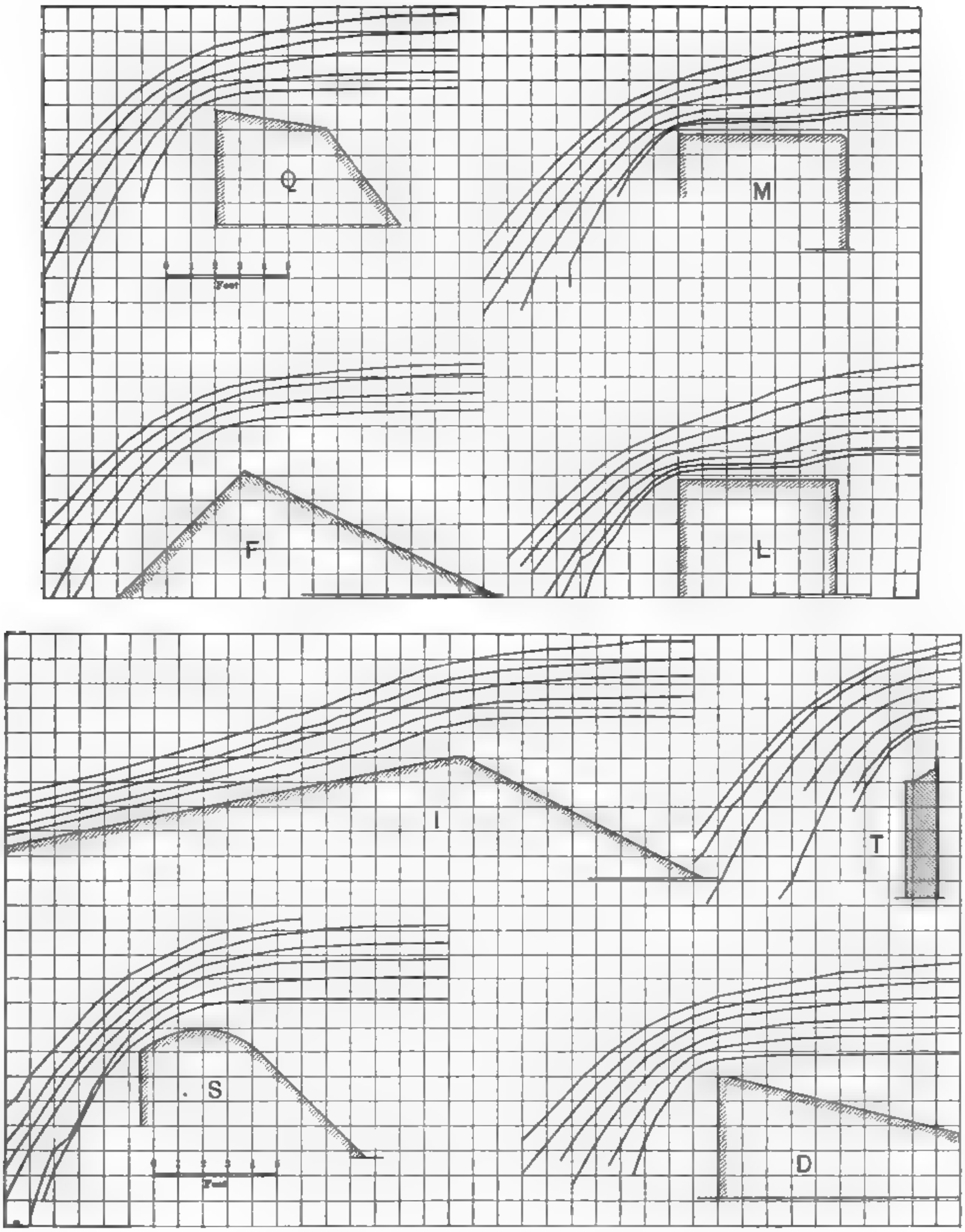


Head
Cms. Ft.
C=3
D=4
E=5
G-H=7-8
J=10
K=11
L=12
M=13
S=19
U.S.D.W.W.
Sharp Edge
U.S.D.W.W. Standard
Discharge { Cubic Meters / Cubic Feet
1.0
35.3
2.0
70.6
3.0
105.9



Head
Ft. Cms.
170
160
5
150
140
130
4
120
110
100
3
90
80
70
2
60
50
40
1
30
20
10
0
C
D
L
M
Sharp Edge
J
K
E
S
H
G
Standard
U.S.D.W.W.
16 ft. long, 6 ft. high
5.0
176.6
6.0
211.9
7.0
247.2
8.0
282.5
DISCHARGE CURVES
OF
VARIOUS WEIR CRESTS.
BASED ON
BAZIN'S FORMULA.
HYDRAULIC LABORATORY, CORNELL UNIVERSITY.
1899.

(To face p. 31.)



PROFILES OF OVERFLOW FOR WEIR-CRESTS OF VARIOUS FORMS.
(Hydraulic Laboratory, Cornell University, 1899.)

The coefficient c has been deduced by experiment and may be found from the following table of values, as given by Trautwine.* In it H represents the still-water height of the upper pool above the crest of the weir, and h is as given above. This formula takes no account of the velocity of approach.

$h \div H$.	Fteley and Stearns' Experiments. (H=0.325 to 0.815 ft.)	J. B. Francis Experiments. (H=1.0 to 2.32 feet.)
0.10	0.625 to 0.635	0.620 to 0.630
.20	.618 to .628	.610 to .625
.30	.600 to .610	.598 to .615
.40	.590 to .600	.586 to .610
.50	.585 to .595	.585 to .607
.60	.583 to .593	.585 to .607
.70	.580 to .590	.585 to .607
.80	.581 to .591	.585 to .607
.90	.590 to .600	....
.95	.610 to .615	....

Velocity of Approach.—*Free Discharge.*—In cases where the velocity of the approaching water is such that it must be taken into consideration, as is usually the case with weirs except when the discharge is low, the foregoing formulas must be corrected accordingly. The following general formula has been supplied, based on the result of the researches of Francis, Fteley and Stearns, Bazin, and others, and applicable to weirs discharging into free air, with or without end contractions:†

$$Q = C'K(l - 0.1nH)H^{\frac{3}{2}}; \qquad (3)$$

where Q = discharge in cubic feet per second;

C' = a constant for a given value of H;

K = a coefficient from Bazin's experiments;

l = length of weir in feet;

H = height of water above crest of weir at the place of actual measurement;

n = number of end contractions.

The following table gives the value of the coefficients, in which

$a = (l - 0.1nH)H$;

A = area of water-section in channel of approach;

$\frac{a}{A}$ = ratio of area of weir-section to area of section of channel of approach;

K = constant for given value of $\frac{a}{A}$;

C' = " " " " " H.

* Engineer's Pocket-book, p. 267f.

† Proceedings of the American Society of Civil Engineers, August, 1900. Discussion by Mr. Walter C. Parmley.

$\frac{a}{A}$	K	H	C'	$\frac{a}{A}$	K	H	C'
0.01	1.0001	0.10	3.580	0.33	1.0599	0.90	3.340
.03	1.0005	0.15	3.520	.35	1.0674	0.95	3.337
.05	1.0014	0.20	3.478	.37	1.0753	1.00	3.334
.07	1.0027	0.25	3.444	.39	1.0837	1.10	3.329
.09	1.0044	0.30	3.420	.41	1.0925	1.20	3.324
.11	1.0066	0.35	3.400	.43	1.1017	1.30	3.319
.13	1.0093	0.40	3.385	.45	1.1114	1.40	3.313
.15	1.0124	0.45	3.376	.47	1.1215	1.50	3.307
.17	1.0159	0.50	3.368	.49	1.1321	1.60	3.301
.19	1.0198	0.55	3.362	.51	1.1431	1.70	3.296
.21	1.0243	0.60	3.358	.53	1.1545	1.80	3.290
.23	1.0291	0.65	3.354	.55	1.1664	1.90	3.285
.25	1.0344	0.70	3.351	.57	1.1787	2.00	3.280
.27	1.0401	0.75	3.349	.59	1.1915		
.29	1.0463	0.80	3.346	.60	1.1980		
.31	1.0529	0.85	3.343				

Submerged Discharge.—Where the weir is submerged and velocity of approach is to be considered formula (2) becomes

$$Qcl=(h+\tfrac{2}{3}d)\sqrt{2g\left(d+\frac{v^2}{2g}\right)},$$

in which v = approximate mean velocity of approach $=\dfrac{3.33\times l\times d^{\frac{3}{2}}}{A}$

and A' = area of water-section above lower pool at the place where d is measured.

Discharge in Open Channels.—On large rivers and in places where a weir cannot be erected for the purpose, the gauging is done by ascertaining the velocity and using it in combination with the cross-section. The surface velocity may be found by floats, and the velocity at any point in a vertical plane by rod- or tube-floats, gauge-tubes, or hydrometers, or current-meters. The velocities change with the depth and width, and those at the surface also vary greatly as the water is deep or shallow. The observations should be made at times of comparative quiet, as the effect of wind upon the floats often leads to inaccurate results.

The mean velocity of a stream in feet per second is obtained by dividing the discharge in cubic feet per second by the sectional area in square feet, and is the average of all the elements of the current. The motion of each particle of water in open channels depends upon the inclination of the surface, two forces being at work, that of acceleration and that of resistance. The former is gravity, the latter friction, which may occur either between the particles of water or on the bed and banks.

The quantity of water passing a given point in a given time can only be found with an approximate degree of accuracy. It is also desirable sometimes to ascertain the quantity which a proposed section of channel having a known slope will pass, and many attempts have been made to find a fixed relation between the slope of a stream and its cross-section, but so far without success. The formulas in use are, of necessity, to a greater or less extent empirical, having been deduced from assumed laws or experiments. It has not been practicable to obtain a formula which can be applied to all

conditions, but two will here be given, the latter of which is considered by many engineers to be applicable to nearly all cases:

D'Aubuisson and Downing's: $V = 100\sqrt{RS}$, in which V is the velocity in feet per second, R the hydraulic radius in feet (found by dividing the sectional area A by the wetted perimeter), and S the slope of water-line. The discharge D, is found by multiplying the velocity by the area, hence $D = VA$. By wetted perimeter is meant that portion of the profile of the section which lies below the water-line.

Kutter's:

$$Q = \left\{ \frac{41.6 + \frac{0.00281}{S} + \frac{1.811}{N}}{1 + \frac{\left(41.6 + \frac{0.00281}{S}\right) \times N}{\sqrt{R}}} \right\} A\sqrt{RS};$$

in which Q = discharge in feet per second;

A = sectional area of channel in square feet;

R = hydraulic radius in feet;

S = slope per foot;

N = coefficient of roughness, its value varying between 0.020 and 0.035 according to the nature of the bed and condition of the channel.

Mean velocities for any position may be deduced from the velocities given by the rod-floats by the following formula of Francis:

$$V' = V[1 - 0.116(VD - 0.1)];$$

in which V = observed float velocity;

V' = mean velocity in the vertical;

$$D = \left(\frac{\text{depth of water minus immersion of rod}}{\text{depth of water}}\right).$$

Humphreys and Abbot in their investigations on the Mississippi discovered that the "ratio of the mid-depth velocity to the mean velocity in any vertical plane is practically independent of the depth and width of the stream, of the mean velocity of the river, of the mean velocity of the vertical curve, and of the locus of its maximum velocity. In other words, it is a sensibly constant quantity for practical purposes." * Later investigations, however, indicate that there is no constant ratio between mean and mid-depth velocity. Neither is the numerical value of the ratio as given by them (that at depths varying from 86 to 27 feet, the ratio has an extreme range only from 0.9868 to 0.9798), constant. These authorities suggested the taking of velocities at mid-depth only, the quantity thus obtained to be reduced to the mean by using the coefficients found.

* Physics and Hydraulics of the Mississippi River, p. 311.

Measurement of Velocity.—By Floats.—Two methods have been in general use for measuring the velocity at any point, that by floats and that by current-meters. For ascertaining the mean velocity in a vertical plane rod-floats are employed, while the velocity at the surface may be obtained by any ordinary floating substance.

Although the method of floats has long been employed, its use has been condemned by certain authorities, especially as regards double floats, because the movement of the lower float is not accurately indicated by that of the one at the surface, and the motion of the lower float, being influenced by a different portion of the current, is modified by the upper one, while invisible eddies may further affect the lower one without indication above.

Surface-floats may be very simple; an orange, apple, cork, piece of wood, or partly filled bottle will answer the purpose when there is no wind stirring. With any disturbance in the atmosphere, however, the floats should be submerged to such extent as to be just visible.

Rod-floats which must be loaded at the bottom, may be made of tin, or may be simple wooden rods. They travel with their tops slightly above the water and their bottoms as near as practicable to the river-bed; but owing to the irregularities which usually exist in the latter, it is not always possible for the lower end to reach the level desirable. Again, since the pressure of a fluid upon a body moving through it varies as the square of its relative velocity, the float will travel slower than the topmost filaments and faster than those at the bottom. This difference being greatest at the latter point, the velocity of the float will be held back toward the bottom, and will be somewhat slower than the mean of the plane; but for measurements in which extreme accuracy is not required this character of float answers very well. It is necessary to make it in sections, which can be fastened together, in order to adapt it to the varying depths of the river.

On certain rivers in America the passage of floats has been recorded electrically.* The apparatus consisted of a wire stretched from bank to bank, and held in position at frequent intervals by short wires fastened to anchors of stone resting on the bottom. A skiff moved across the river, held by the wire and provided at the stern with a strip of wood 20 feet long, placed normal to the current. From each end of this strip a wire about 100 feet long and supported on buoys trailed down stream, having its other end fastened to a similar strip, this being provided with a slack copper ribbon. The floats were started from the skiff, and on striking the ribbon they closed a circuit and rang a bell in the skiff. They were then caught by a man in another skiff and returned.

Double Floats are of several forms and are made of wood, tin, galvanized iron, or other materials. A simple one is made of two jugs connected with a string. The idea is to attach to a surface-float, by a fine cord or otherwise, a submerged float, which may be adjusted to various depths. The bottom float should be of such a weight as will keep the connecting cord tight without drawing the surface-float wholly out of view.

* Annual Report, Chief of Engineers, U. S. A., 1883.

The theory of the movement of this class of floats is that the lower one will be carried along by the water in which it is situated, while that at surface will travel fast or slow in proportion as the current there is faster or slower than that below. This method is applicable to large streams as well as to those of great depth or with rapid currents, but it gives only the approximate velocity of that portion of the water through which the lower float travels. To obtain a velocity at a point between the upper and the lower floats it will be necessary to shorten the connecting cord and make a new observation.

For reasons mentioned, double floats have not proved very satisfactory in use and the opinion of experienced observers is that they can rarely be relied on within 10 per cent of the truth. Individual results have been known to vary 25 per cent on similar observations. Moreover, in high water or swift currents the effects of eddies and of cross-currents in changing the velocities of the floats is very noticeable.

By Current-meters.—Current-meters have to a great extent succeeded floats for the gauging of streams, but they have not proved wholly satisfactory. Much of the dissatisfaction with them undoubtedly comes from their mechanical construction and from careless use and rating. The rating is done by moving the meter through quiet water, at a depth of about 5 feet, and at a uniform rate of speed. Its purpose is to obtain the ratio between a revolution of the wheel and the velocity of the current. These ratings should be tested frequently. The meter is well adapted to streams of good size which are comparatively free from drift, and in which the current is neither too sluggish for certain action nor too swift to secure proper anchorage. The principle is that of obtaining the velocity of the current by counting the number of revolutions it will produce on a wheel, screw, or vanes, properly arranged. The revolutions may be recorded by a system of toothed wheels in the instrument, or by an electrical apparatus protected from the water and usually placed on shore. The meter is operated from a boat anchored where desired, and is weighted in order to lower it to the depth required.

To find the mean velocity in a vertical plane on the cross-section the meter is lowered at a uniform rate of speed from the surface to the river-bed, and then raised again to the surface, taking the reading for the bottom and for both surface positions. Each of these is divided by the time of movement and a mean of the results taken.

By Gauge-tubes.—Pitot used a glass tube, open at both ends, and having a short horizontal and a long vertical arm connected by a short bend. By immersing the tube with its long arm upright, the end of the horizontal portion being presented to the current, the pressure will raise the water in the tube and thus furnish a measure of the velocity. An improvement was made upon this apparatus by Darcy, who employed two tubes connected at the top by a copper tube containing a stop-cock, and provided at their bottoms with bent copper tubes with mouth-pieces and a double-acting stop-cock. Before placing the apparatus in the water the upper cock is closed and the tubes are then immersed to the depth desired, with one mouth-piece to the cur-

rent. The other mouth-piece is at right angles to it. The lower stop-cock is then opened and the water rises in the tube which points to the current. The cock is then closed. The water will stand at a higher level in this tube, and this is the measure desired, and is represented by the formula $V = \mu\sqrt{2gh}$, where h is the height of the column, and μ is a coefficient to be determined by experiment.

By Hydrodynamometer.—The engineer Gros de Perrodil designed an instrument for measuring the current by means of the torsion produced by the pressure against a disc upon a wire placed vertically in the water. The velocity was obtained by the equation $V = c\sqrt{a}$, in which $a =$ the angle of torsion, the value of c being found by experiment.

Sediment.—The determination of the quantity of sediment carried in suspension is often of great importance in the study of river improvements, particularly where a decision is to be made between fixed and movable dams, as when silt travels in great quantities its deposit in the upper and shallow parts of pools formed by stationary dams will often seriously interfere with navigation, and may even permanently raise the river-bed. It is also of importance in the construction of works of contraction. Formerly the plan of improvement of the Mississippi was considered to be wholly dependent upon the understanding of the laws connected with the suspension and transportation of the sediment.

It is necessary in these observations to secure samples of water from various depths below the surface and at different points of the section. These are allowed to settle and the sediment in each is then weighed, a proportion being thus established. For this purpose a tin can may be used attached to a graduated pole and having a valve at the bottom which is opened by a string leading to the top of the pole, and which closes by its own weight. For great depths it is necessary to weight the can and use a cord instead of a rod.

Some interesting data upon the subject of sediment are given in connection with the Arkansas River, in a pamphlet by S. C. Branner, as follows: *

"The matter in suspension is greatest during a sudden high rise; but after the water in the stream stands at any high mark for a few days, the decrease of the amount of suspended matter it carries is very marked. The amount of sediment carried by the river varies widely also with the same gauge-reading at any stage, being greater with a rising and less with a falling river.

"The greatest amount of sediment found in the water during the year under consideration was 225 grains to the gallon ($\frac{1}{259}$) when the river stood at 17 feet on the gauge, and after protracted rains. (Extreme high water at Little Rock is about 28 feet.) It should be added, however, that while this high water may be taken as a type of the ordinary rises, there are times when there is but little or no rise, no increase in the volume of water discharged, but a very marked increase in the amount of mechanically suspended matter. In October, 1891, occurred one of these so-called 'red rises'

* Observations upon the Erosion in the Hydrographic Basin of the Arkansas River.

of the Arkansas River, and although the river was quite low,—marking only 3.9 feet on the gauge,—it carried 761 grains of matter to the gallon, of which only 46 grains was matter in solution (that is, the matter in suspension was $\frac{1}{87}$ by weight).

"The matter in solution bears no constant relation to the volume of water, though in a very general way it varies inversely with the volume of the water, and ranges from 11 to 70 grains to the United States gallon. The amount carried down, in this form, from October, 1887, to September, 1888, was 6,828,350 tons. During the single month of May, 1888, 1,161,160 tons were carried out in solution. Taking the observations for the entire year under consideration, the matter in solution is equal to about 0.31 of that in suspension. These relations, however, are not constant. In November, 1887, for example, the dissolved matter was more than six times as much as the suspended matter—while on October 13, 1891, the suspended matter was more than thirteen times the matter in solution."

(3) SLOPE.

The slope of a river may be ascertained by leveling or by gauges set at known elevations and read simultaneously. It varies greatly in different parts of a river and at different stages in the same part, and even at similar stages at different times. Local conditions govern it, and these conditions are always changing in rivers with friable beds. When the water is low, the profile presents a broken outline; as it rises this gradually changes, until at flood height the slope may be nearly uniform for long distances, because it is not so closely affected by the bottom and by the banks.

Gauges for Local Slope.—The method of ascertaining the slope for use in gauging operations is to make simultaneous readings, say at five-minute intervals, on a series of gauges erected at regular intervals of say 500 feet along the shore, for distances of a half mile above and below the range-line or gauging section. The gauges are usually graduated to half tenths and can be read approximately by means of a vernier to thousandths. They are inclosed in boxes which reach down 2 or 3 feet below the water surface, and the water is admitted through a hole in the bottom, thus securing the gauge from wave action. Where observers are not available for each gauge, the readings are taken at longer intervals by a man passing along in a skiff as rapidly as possible from the upper to the middle gauge, and another man from the middle to the lower gauge, the middle gauge being read by each man and corrections made on all readings as necessary.

Observation of Daily Stages.—The recording of the stage of water in a river at various points at the same hour every day will furnish useful information to those who are engaged in the study of its improvement. For a complete and continuous record of tidal stages there is in use a self-registering, automatic gauge, but as a general thing observations on rivers are made by means of ordinary gauges, graduated in feet and tenths, and placed with their zeros having reference to some established datum. These gauges may be attached to lock-walls, bridge-piers, trees, or other fixed objects, and

should be so placed that their positions cannot be disturbed by passing craft, drift, or other causes. The observations should be made by reliable and careful persons at the same hour each day at all stations, and in addition to the stage of water they should note whether the river is rising or falling, and if an extreme of high or low water has been reached during the 24-hour period the time and stage should be reported.

Datum for Zeros.—When the object of the gauge is to record heights of water for the use of navigation, the zero is usually placed at the level of the lowest water known. If it is to show the depth of water on a certain bar, shoal, or other point, usually the point of least draught in a certain locality, its zero is fixed at the depth below the water surface which will indicate the depth on the point in question. For instance, if the depth is 3.6 feet on the head of Mustapha Island, in the Ohio River, the gauge at Parkersburg, 11 miles above, should read 3.6 feet, Mustapha being the governing bar of that section of river. A pilot then knows at a glance whether he can proceed, because the stage on the governing point indicates to him that all other points in the section traversed have at least as much water as that shown by the gauge.

Material for Gauges.—*Wood.*—The simplest form of gauge is a wooden board, painted and properly graduated, but this is only of temporary duration. Their renewal is necessary at frequent intervals because they are easily destroyed, and their decay is certain. Their replacement is not always effected with absolute certainty as to the zero being placed at the same height as before. They must be frequently painted, and a derangement of their divisions may occur in this manner. A gauge once properly established should remain without change or disturbance, and with wooden gauges this is not practicable.

Metal.—Cast- and wrought-iron and steel gauges are in use and give fair satisfaction. Those of cast iron usually have their figures and division lines raised, the body of the gauge being painted white and the lines and figures black. Usually these gauges are cast in sections from three to six feet in length, with suitable ears for fastening them in position. These short sections facilitate the repairs which are sometimes necessary, on account of breakages. On the upper Rhine the cast gauges have raised enameled figures and divisions, and it is said the enamel does not scale off as it does upon flat surfaces.

Gauges of wrought iron and steel have been long in use, the figures and lines being stamped and enameled. This form readily cleans itself and preserves a clearness of color for a time, but the enamel is easily broken and scaled off, particularly on broad surfaces where expansion plays a part, thus causing the figures and divisions to become indistinct and the reading difficult and uncertain. This class of gauge is also rather expensive.

Stone.—Gauges of stone are not uncommon in this country. They are sometimes laid on the slope of the bank and form a walk. The stones are cut and laid carefully, precautions being taken to prevent undermining either on the edges or at the foot. The division lines and figures are then located with a spirit-level and cut into

the stone. Where the gauge is to be utilized as a walk the stones are usually from six to ten inches thick and three feet to four feet wide. The figures are cut at one side so as to avoid wear from travel. Where the gauge is not to be used other than as a gauge the stones are generally about six inches thick by two feet wide, and are set on their edges, projecting above the ground a few inches only. Gauges of this kind should be located at points where they will not be covered by the settlement of sand, mud, or débris. They are expensive, and are ordinarily not resorted to where vertical objects can be obtained for fastening other forms. Wooden gauges placed on slopes in the same manner are also in use.

Wall Gauges.—A very common method on improved rivers is to cut the gauges on the lock-wall masonry. A strip of wall about a foot in width is made smooth and slightly countersunk behind the general face of the wall. In this are cut V-shaped division lines and figures, the latter being usually Roman characters, about one-half inch deep at their center lines. The lines and figures are painted black and the face of the recess or strip white, and the result gives a very distinct gauge. It is, however, like most gauges, open to the objection of being difficult to keep clean and bright, besides which, the paint comes off more easily than from wood.

Tile.—A new kind of gauge has come into use within the last few years which bids fair to be more satisfactory than those heretofore employed. It is composed of vitrified tile blocks, six inches square, and from one-half to three-quarters of an inch in thickness inlaid in different colors to represent the scale and numbers. The blocks are sized with care, and if set against a masonry wall with Portland cement will adhere firmly. As they are impervious to moisture, the faces can easily be kept clean, and the colors will not fade or scale off. Where used on locks, they are set in a recess in the wall, the face of the tile being brought out nearly to the face-line of the wall.

PART II.

IMPROVEMENT OF OPEN RIVERS.

CHAPTER I.

REMOVAL OF BARS AND OTHER OBSTRUCTIONS.

Desiderata.—The conditions to be fulfilled in the betterment of navigation in open rivers are:

1. A sufficient depth for navigation at all seasons.
2. A sufficient width of channel for safe passage of all craft traveling in opposite directions or in the same direction.
3. The works necessary to secure such results must not be of an obstructing character, nor must they cause the formation of obstacles to navigation, nor lower the water-level sufficiently to uncover those in the river-bed.
4. The cost of construction and maintenance must not be out of proportion to the benefits to be derived.

Expedients.—The fact that it may cost a larger sum of money to render a river satisfactorily navigable by employing the best methods often leads to the acceptance of makeshift or temporary forms of improvement. This has brought about the application and suggestion of numerous plans and contrivances, some with merit, some wholly impracticable, and usually all undesirable where it is important to attain permanent and satisfactory results. The temporary contrivances used for this purpose, or suggested for trial, are of great variety, and consist of trellis-work, basket-work, movable gates, swinging mattresses, etc., for deflecting the current, and scrapers and stirring devices of various designs for loosening the bars and assisting the current in their removal.

The principal trouble with all these appliances, however, arises from the fact that the current cannot be relied upon to carry away the material after it has been loosened, notwithstanding the general belief that when the crust of a bar has been broken it is an easy matter to wash it away. Janicki has called attention to one effect produced by them in these words: "Such machines have a moral effect, if I may be allowed the term. Boat-owners, who have to suffer from low water on the bars, complain less if they see that something is being done to relieve them."

NOTE.—A number of the illustrations are taken from a paper by J. A. Ockerson on "Dredging in the Mississippi," Trans. Am. Soc. C. E., vol. xl.; others are furnished by the Bucyrus Co., South Milwaukee, Wis.

The principal of these devices are as follows:

Gates.—M. Borrel and M. Collin * applied upon the Garonne and the Loire, respectively, more than a half century ago, movable gates which were so placed as to contract the current over a bar and produce scour by the increased velocity. Thus a passage for boats was created by a simple displacement of material. The gates were reset as often as required.

M. Fouache used a trapezoidal leaf having teeth along its lower edge to deepen the Somme Canal. It was supported at the desired height by boats, so as to confine the stream and thus increase the water-level above and reduce it below. The head thus produced moved the gate along the canal, and with it the material torn up by the teeth as the leaf traveled. The depth of the cutting-edge was regulated by windlasses on the boats.

Engineer Masquelez, in 1811, used a similar device in building the canals at the mouth of the Charente. It was placed astern of a suitable boat, the bottom being loosened with a hook. Two movable wings served to close the canal behind the boat and thus produced the head necessary for movement, the bottom of the canal being cut as with a plane and pushed along toward the outlet.

Screws.—"About the earliest application of this principle on the Mississippi was in 1867, when it was decided to improve Pass à l'Outre by means of excavating and stirring up the alluvial material deposited from the heavily laden waters of the river.† In this work a double-ended dredge-boat, having an excavating-screw with four blades 14 feet in diameter, was used. This screw was similar to an ordinary propeller-wheel and was similarly mounted. It was turned by means of a double engine at the rate of 60 revolutions per minute, and reached a depth 2 feet below the keel. The work of the screw was made more effective by auxiliary scrapers attached to the up-stream end of the boat, on each side of the keel. The boat was moved down stream over the bar with the screw operating and the scrapers in position. In this way some of the bar material was again brought into suspension and carried off into deep water by the current.

"During the first month's work with this dredge the depth was not materially improved. Later, better success was realized, and in a little less than two months the depth was increased from 11 to 17 feet. The chief difficulty seemed to be in weak propeller-blades, which were frequently broken and could only be renewed by docking the vessel. This device was intended to cut out and maintain a 20-foot channel through the bar at the mouth of the river.

"At the Southwest Pass the same result was expected from the use of conical screws attached to the bow of a suitable boat. These cones were 20 feet long and 5 feet in diameter at their bases. They were set so that their points came together at the boat's stem, and their bases were separated so as to cover a width of 20 feet from out to out.

* Annales des Ponts et Chaussées, 1835.

† Trans. Am. Soc. C. E., vol. xl, pp. 223-226, J. A. Ockerson.

General View of a Two-yard Dipper Dredge, with Wire-rope Hoist.

(The illustrations of dredges are reproduced by courtesy of the Bucyrus Dredge Co.)

(To face p. 43.)

Their axes were horizontal, the salient angle being foremost. The flanges of the screws were 12 inches wide at the base of the cones, and diminished to 6 inches at the points. When these enormous screws were put in motion it was very difficult to guide the boat. The material was readily plowed up, but it was not broken sufficiently fine to be carried away by the current.

Scrapers.—"In 1867 there was appropriated the sum of $96,000 for the construction and operation of two scrapers or dredges on the upper Mississippi, between St. Paul and the mouth of the Illinois River. The first efforts made to remove the sand-bars by means of the scrapers, which were invented by Col. Long, was in the fall of that year. These scrapers consisted of a frame attached to the bow of a boat and carrying a heavy cross-bar, to which were attached six steel buckets or cutters. The frame could be raised or lowered at will. In operating, the boat went to the upper side of a reef, the scraper was lowered, and the boat was backed slowly down stream, scraping the sand with it to the deep water below the reef. This operation was repeated until the desired depth was obtained. Two side-wheel steamboats were equipped with these scrapers by the Government, and, for a time, steamboat owners operated a scraper boat between Keokuk and St. Louis at their own expense.

"One boat was equipped and ready in October, 1867. Her first work was on a bar near Gray Cloud, 17 miles below St. Paul. Only 3½ feet of draught could be carried over this bar, and the regular packets could not cross it. After about four hours' work with the scraper the depth was increased to 3½ feet entirely across the bar. The scraping was continued for two days and a depth of 4 feet was secured. By November 15th all the bars between St. Paul and Prescott had been scraped and the depths increased to 3½ or 4 feet. At that date the packet companies notified the engineer in charge that the scraping had removed all obstructing bars and that no more work was required.

"In 1868, when navigation again became difficult, the scrapers were put into commission and worked throughout the season. They succeeded in deepening the bars from 8 to 18 inches, and this was generally accomplished with a few hours' work. Beef Slough was deepened from 3½ feet to 4½ feet in 35 minutes.

"On the whole, the results were so satisfactory that steamboat owners announced that their boats had been making regular trips without interruption, 'a condition of affairs never before known at this stage of river in the experience of pilots of thirty-five years' standing.' The largest steamers had been able to reach St. Paul in the low-water season during two successive years, when without the aid of the scrapers they would have been obliged to tie up.

"This scraping was continued for several years at a cost of about $20,000 per annum for each steamboat, but, as the relief was only temporary and had to be repeated from year to year, it finally gave place to the so-called permanent improvement, consisting mainly of channel contraction.

"It should be borne in mind that in the portion of the river where the above-described scrapers were used the obstructing sand reefs are quite short."

Jets.—In the various inventions brought forth the water-jet has had a prominent place, the object being to enable a steamboat, upon running onto a bar, to work its way through by means of pumps, and some of these devices have met with considerable success. In 1881, for example, a bar near St. Louis was cut through by the use of pumps mounted on boats, and having a capacity of about 165 gallons each per minute. In ten hours the channel was deepened from 6 feet to 8.3 feet, and made wide enough for the largest tows.

A special jet-dredge was built in 1896, for use on the Mississippi between St. Louis and Cairo, the pumps being two 15-inch centrifugals, each with a capacity of 10,000 gallons per minute. On short bars the work was very effective, but on long ones the piling-up of the sand in front of the jets prevented successful results.

Dredging.—Dredging as a means of river improvement has been extensively resorted to in all parts of the world. It is in the harbors and mouths of streams, however, that it has had its widest application. With the exception of the lower Mississippi, where it is in use as a means of giving temporary relief during low-water seasons, its employment in this country has not been on an extended scale on non-tidal streams. In connection with blasting it has been found quite efficacious on some of our best rivers, and where the river-bed is of a fairly hard material the results from dredging have been generally satisfactory. However, in the majority of cases, redredging has been found necessary from time to time in order to maintain the desired depth, this being the great objection to this process of improvement. When the material is taken out it must have a place of permanent deposit, or it will be washed into chutes lower down stream, and the best results from dredging are only attained in those streams and those places where there is little drifting material. A bar at the mouth of a creek may be removed each year, and still navigation will be impeded by new material coming into the cut excavated. This process of improving navigation is not confined to open rivers, but is carried on to a considerable extent in those streams having works of canalization.

Types of Dredges.—The following are the principal types of dredges in use:

Dipper Dredge.—This consists of a boat provided with an iron dipper or bucket, secured to the end of a long handle, which is mounted on a revolving boom so as to permit a wide range of digging. To operate it the dipper is lowered on to the bed of the river, and pulled along in an almost horizontal direction till it has scraped up a load. It is then raised (the handle being arranged to slide up and down and held wherever desired by friction), the boom is swung, and the load dumped into scows or elsewhere by opening the hinged bottom of the dipper. The boat is held in position by "spuds," or long timbers, which move vertically in sockets in the hull, and are let down onto the bottom before digging. Three spuds are provided, one on each side of the bow, and one at the middle of the stern. For digging in hard-pan or similar substances, large teeth are bolted to the mouth of the dipper, serving to break up the material.

General View of an Elevator Dredge Arranged for Shore Discharge.

(The illustrations of dredges are reproduced by courtesy of the Bucyrus Dredge Co.)

(To face p. 45.)

These dredges are at present made of sizes from one-half to eight yards in capacity, and to operate in depths of water up to 40 feet. They require a considerable amount of space in which to work, owing to their method of operation, and in this respect they are inferior in certain situations to the clam-shell dredge. They will, however, excavate almost any material except solid rock.

Clam-shell Dredge.—This style is similar in general arrangements to the dipper dredge, but has a clam-shell bucket instead of a dipper. This consists of a bucket shaped like a semi-cylinder, hinged so it will open into two parts. It is suspended from the end of the boom, being kept from twisting by two long poles which work up and down through eyes on the boom. The operation is done by two chains, one of which lowers the bucket rapidly in an open position so it will penetrate the river-bed, while the other is arranged to pull the halves together, thus scraping up the load, and then hoists the bucket to the surface. The load is dumped by slacking this chain and holding the other, when the bucket opens again as in its first position. Frequently a hemispherical shape, divided into four parts, is used instead of a semi-cylindrical one divided into two; the bucket is then called an "orange-peel" or a grapple. This style is more serviceable for general use than the other.

Formerly a type was made with only one chain, which pulled the parts together, latches being employed to release the load. It was found, however, that when the blades caught on any immovable substance, such as a buried log or a projecting rock, it was almost impossible to get them loose, as the chain only acted for closing, and for this reason the type has been abandoned.

The clam-shell dredge will only operate in loose material, such as gravel, blasted rock, etc. For hard or tenacious clay it is of little value. It has the advantage, however, of being able to work in confined positions, such as in sinking cylindrical caissons, and also of being suited to great depths, as there is little straining on the spuds, such as occurs with the dipper dredge.

Combination Dredge.—For miscellaneous work the dipper and clam type are sometimes combined, the boom and machinery being so arranged that either style of bucket can be used. This is sometimes very convenient for river work, especially in dredging in lock-chambers.

Elevator Dredge.—The type of dredge generally employed in England and other parts of Europe is that known as the elevator or bucket dredge. It is used in Canada also, although it has not met with much favor in the United States. With it tolerably hard materials may be excavated to a considerable depth.

It consists of two parallel endless chains, carrying a number of buckets, which are successively presented in a horizontal position to the soil to be excavated by means of an arm swung at its upper end from a frame, while its lower end rests on the river-bed. The chains pass around drums at the two extremities of the arm and are driven by an engine. The arm may be adjusted to suit the depth of water, etc., and may be vertical or inclined. As the material is brought up it is discharged into a hopper or

chute, sufficient water accompanying it to wash it to such a point as desired, or to load it into scows. Some of these dredges are arranged for self-propulsion, and the material is then dumped directly into a hopper in the boat itself. When this is full, dredging operations cease and the boat is run to the dumping-ground and the material deposited. This process has the advantage of causing no lost time, the same crew doing all the work, but it is evident that it is not as speedy as when dredging goes on constantly, with a separate crew attending to the deposit of the material excavated. In working, the dredge is moved slowly over the part of bottom being worked upon in order that each bucket may be filled.

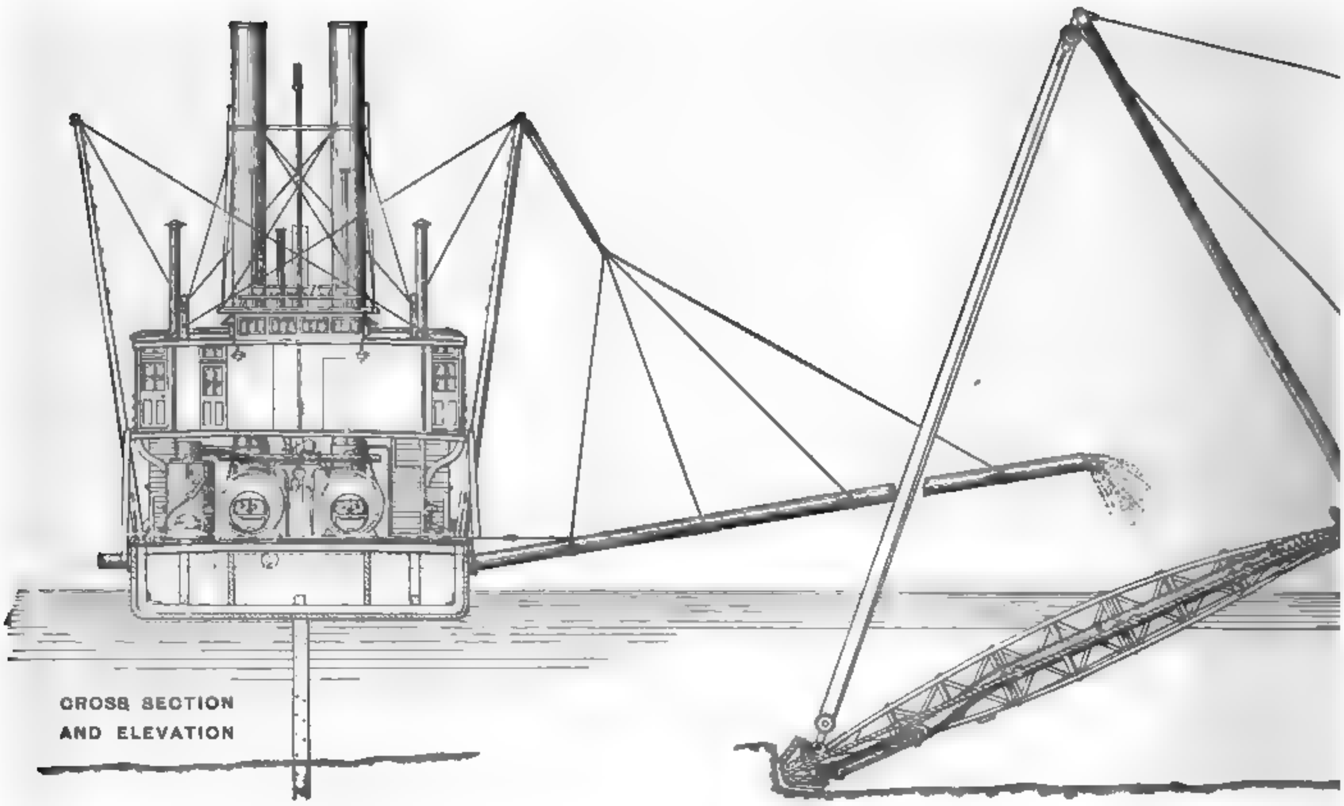
As a general thing the bucket-ladder is located along the center line of the hull, but in some it is situated along the side. The type is also built with two sets of buckets or two elevator systems.

Two dredges of the elevator type were used on the lower Mississippi in 1888, and did very efficient work. One of them is reported to have dredged 4000 cubic yards in a 10-hour day.

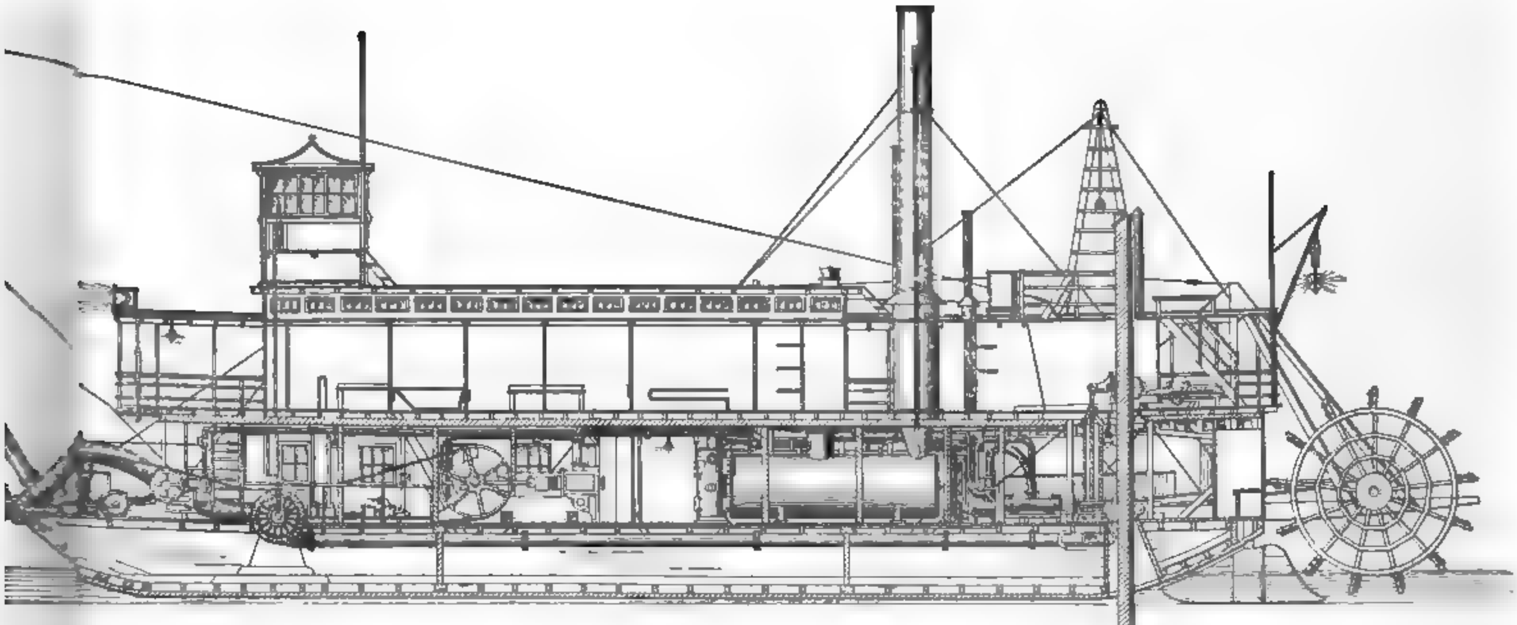
Rock Dredge.—There was used in France as far back as 1852 a dredge for excavating rock. A somewhat similar apparatus was also used on the Suez Canal, and later at the rapids of the Mississippi, at Rock Island, and at the Iron Gates of the Danube. This machine has a series of chisels or pointed rams, about 8 inches square and 16 feet in length, and weighing from 4 to 10 tons, which are run in leads and let fall from a height on to the rock. Behind the chisels in some of these machines is arranged an endless chain with buckets for removing the rock when broken. In others the ram is separated from the dredge proper, the one cutting and breaking the rock into pieces while the other removes it.

Hydraulic Dredge.—A great amount of the dredging in the rivers of America consists of sand and silt, which it is possible to remove by means of pumping. This is done by means of dredges fitted with centrifugal pumps having suitable suction- and discharge-pipes, the largest ones of this type being in use on the Mississippi River. Their duty consists in opening up channels in low water through sand-bars, the sand being pumped up and discharged through a long length of pipe to where it will not wash into the cut again. The work has, of course, to be repeated every season, and sometimes more than once during the same season. Experience, however, shows that, at least in some of the chutes, the material filled in is looser than the older river-bed, and that in the succeeding season the old channel may be reopened by the river because of the greater ease with which the new material can be cut out.

Mississippi River Dredging.—The method adopted, after considerable experimental work, for improving low-water navigation on the Mississippi River was by means of hydraulic dredges of large capacity, which would open a channel through a bar, within a short time, sufficiently wide and deep to accommodate navigation, and to a considerable extent, to direct the flowing of the current along the line of least resistance. The width of this channel is usually 250 feet and its depth 9 feet. The Commission having the



DREDGE RAM



LONGITUDINAL SECTION AND ELEVATION

(To face p. 46)

General View of a Twenty-inch Hydraulic Dredge.

(To face p. 47.)

matter in charge has reported that the plan has met with such success as to justify the continuance of dredging. They claim that it has proved to be "a successful, economical, and reliable means of low-water channel improvement."

The report of the Mississippi River Commission for 1900 showed that during the season of 1899 five dredges were employed. They cut about 62 miles of channel, averaging 105 lineal feet per hour. The channels were maintained without difficulty, the season being very favorable.

Location of Chute.—In this work much depends on the skill with which the location of the channel is made and also upon the water conditions. When the water remains stationary, or steadily falls, a channel once opened remains navigable as long as there is low water. If the location is along lines that the current in seeking a crossing will readily follow, it will not only remain open but will steadily improve. No established rule can be laid down, however, for the location of this channel, it being necessary to make a study of each case as it comes up and then so to place the dredge that it will do the work effectively. The plan pursued in recent years has been to rapidly survey, some time before extreme low water, the places most liable to require deepening. A second survey is made at the time it is desired to begin operations. A study is then made of the results of the surveys, followed by further examinations and observation, and a decision is reached as to where the cut should be undertaken. Sometimes it becomes necessary to abandon a chute after it is started, because the conditions prove unfavorable to its maintenance, while in some of the channels excavated it is necessary to remove additional material from time to time.

Operation.—To operate one of these dredges two wrought-iron anchor piles are sunk by water-jet about 25 feet apart, and about 1000 feet above the point where the dredging is to begin. Wire cables are run to them from the dredge, which then commences pumping, slowly winding in the cables by steam-drums, the rate being of course commensurate with the capacity of the pumps, an average being between 60 feet and 80 feet per hour. In windy weather it is necessary to set side piles to steady the boat. After one cut is finished the piles are moved over for another cut, the dredge dropped down stream, and the operations recommenced. The excavated material is deposited through the discharge-pipes, several hundred feet away.

The piles consist of hollow wrought-iron tubes, closed at the top, and open at the bottom, with an attachment near the upper end for a 2½-inch pressure hose. They are sunk 15 feet to 20 feet into the sand, the mooring lines being attached to shackles near the river-bed. A special boat carries the apparatus used for sinking them.

In busy seasons the dredges are run 24 hours per day, and the cost per cubic yard is given as from four-fifths of a cent to fifteen cents, depending on the looseness and quantity of the material. The cost of the dredges is from $90,000 to $110,000, according to size.

Dimensions of Dredge.—The dredge Iota, which is one of the most recent of the Mississippi River fleet, has a hull of steel, 44 feet by 192 feet by 7 feet deep, and is self-

propelling, being provided with a pair of side paddle-wheels 21 feet in diameter. The draught was designed to be 48 inches. The boilers are seven in number, set in three batteries, and with a working pressure of 170 pounds. The pumping outfit consists of a centrifugal pump with a 32-inch discharge, capable of delivering not less than 1000 cubic yards of sand per hour through 1000 feet of pipe, and is operated by a pair of horizontal tandem-compound condensing engines of 16-inch and 26-inch cylinders, and 20-inch stroke, direct connected. The sand agitator is of the water-jet type, acting under a pressure from a duplex pump of 60 to 100 pounds per square inch. The pipe-line is of ¼-inch steel plate, in 50 feet sections, supported on pontoons and with swivel joints. Twenty anchor piles, 35 feet long and 11 inches outside diameter, of metal from ⅝ inch to ¾ inch thick, were supplied with the dredge.

Full quarters were constructed on the hull, including laundry, bath-room, machine-shop, and refrigerating and electric-light plants.

Dredges for Improved Rivers. — The best type of dredge for a river possessing locks and dams, where these are sufficient in importance to require annual service of this nature, is the dipper dredge, fitted for use with a grapple bucket also. Its capacity should be not less than 2 yards, as small dredges are very uneconomical, costing almost as much in operation as large ones. The machinery should be of special power, as part of the work will consist in tearing out wrecks, snags, old cribs, etc., and the strength of all parts should be designed accordingly. The limiting depth of water in which to work should be 18 or 20 feet, as the dredge will then be practically independent of summer rises, and can work on uninterruptedly. This is sometimes a matter of much importance, as we have more than once seen important works delayed in times of pressure because a rise of a few feet put the dredges out of operation, their limit of depth being 12 to 15 feet. The coal capacity should be ample, so that the work can be carried on for several weeks away from the base of supplies.

In the last few years steel-hulled dredges and scows have come into use, and have given excellent satisfaction. They are much stiffer than the wooden hulls, and give no trouble from leakage, and if they are occasionally painted appear to outlast two or three of the usual type.

The scows should always be of the side-dump variety, so they can be used for backing dams, filling washouts, etc. The bottom-dump is usually inapplicable for this, as it needs a depth of water of 8 to 10 feet for dumping.

Dynamite.—Dynamite is principally employed for the removal of snags, rocks, etc. Its use for assisting in the removal of sand-bars has often been proposed, but seldom applied, and then with uncertain measure of success. It was used on the Brunswick Bar in Georgia from 1891 to 1895, and also at Aransas Pass, Texas. At the former locality about 100,000 pounds were used, with the result as shown by surveys that the four years' work had only deepened the channel 1.7 feet. Dredging was then resorted to, and a further depth gained by this means of 1.6 feet in eight months, or almost as much as in the whole four years preceding.

SEA-GOING DREDGING STEAMER *Reliance.*
(New York Harbor, 1890.)
Length, 157 feet; Beam, 37 feet; Depth, 16 feet; Carrying Capacity, 650 cu. yds.

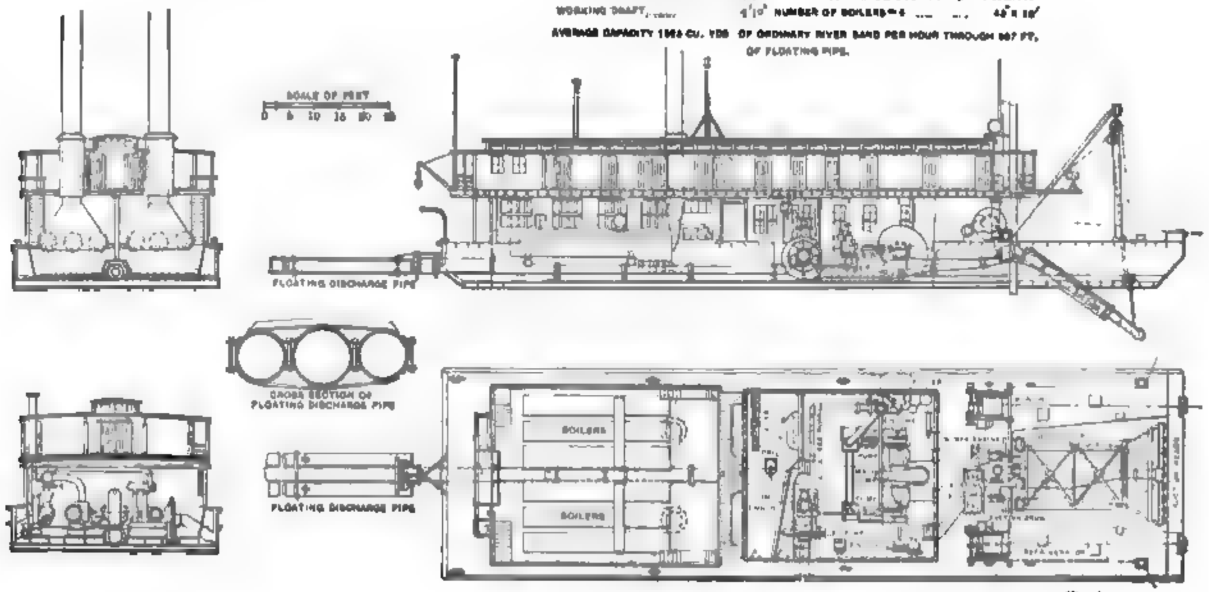
(To face p. 48.)

HYDRAULIC DREDGE GAMMA

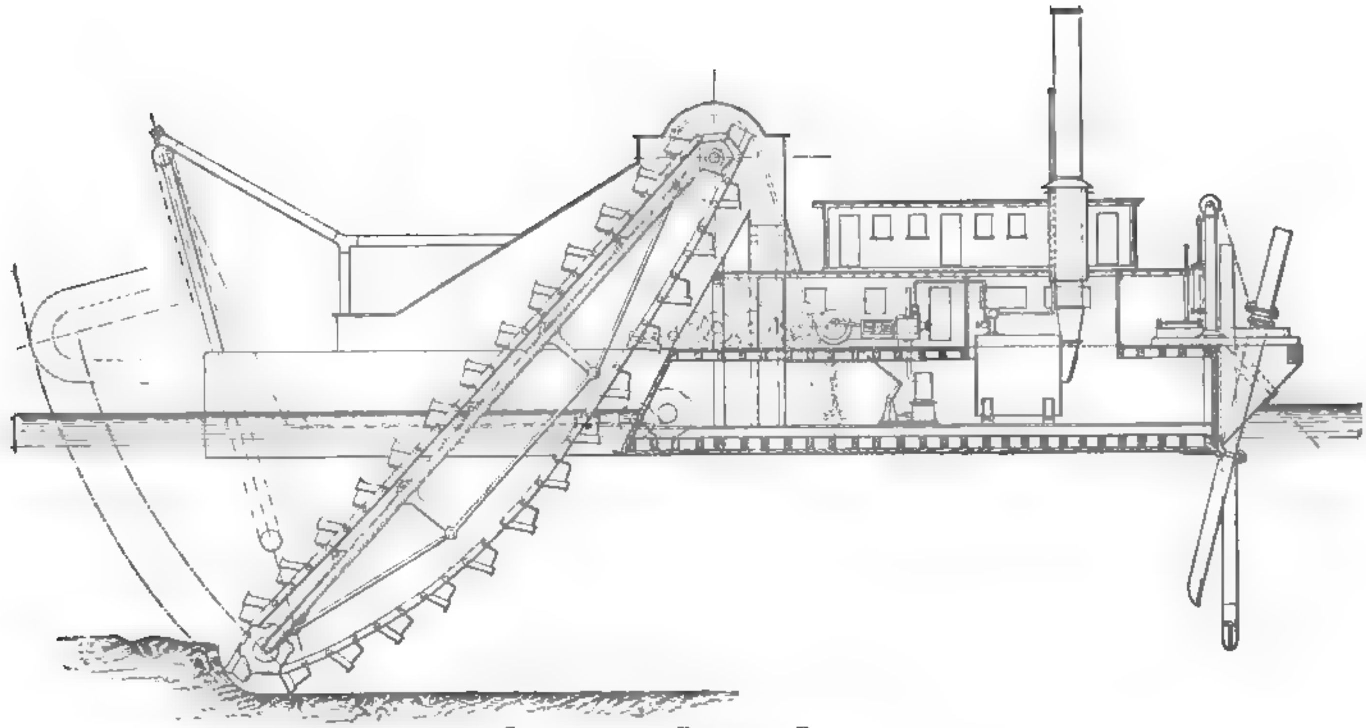
DIMENSIONS.

LENGTH OF HULL	[illegible]	DIAM. OF CENTRIFUGAL SAND PUMP	[illegible]
LENGTH OVER ALL	[illegible]	DIAM. OF SUCTION AND DISCHARGE PIPE	[illegible]
BREADTH OF HULL	[illegible]	LENGTH OF DISCHARGE PIPE	[illegible]
DEPTH OF HULL	[illegible]	DIAM. OF CENTRIFUGAL JET PUMP	[illegible]
WORKING DRAFT	4' 10"	NUMBER OF BOILERS—4	48" x 16'

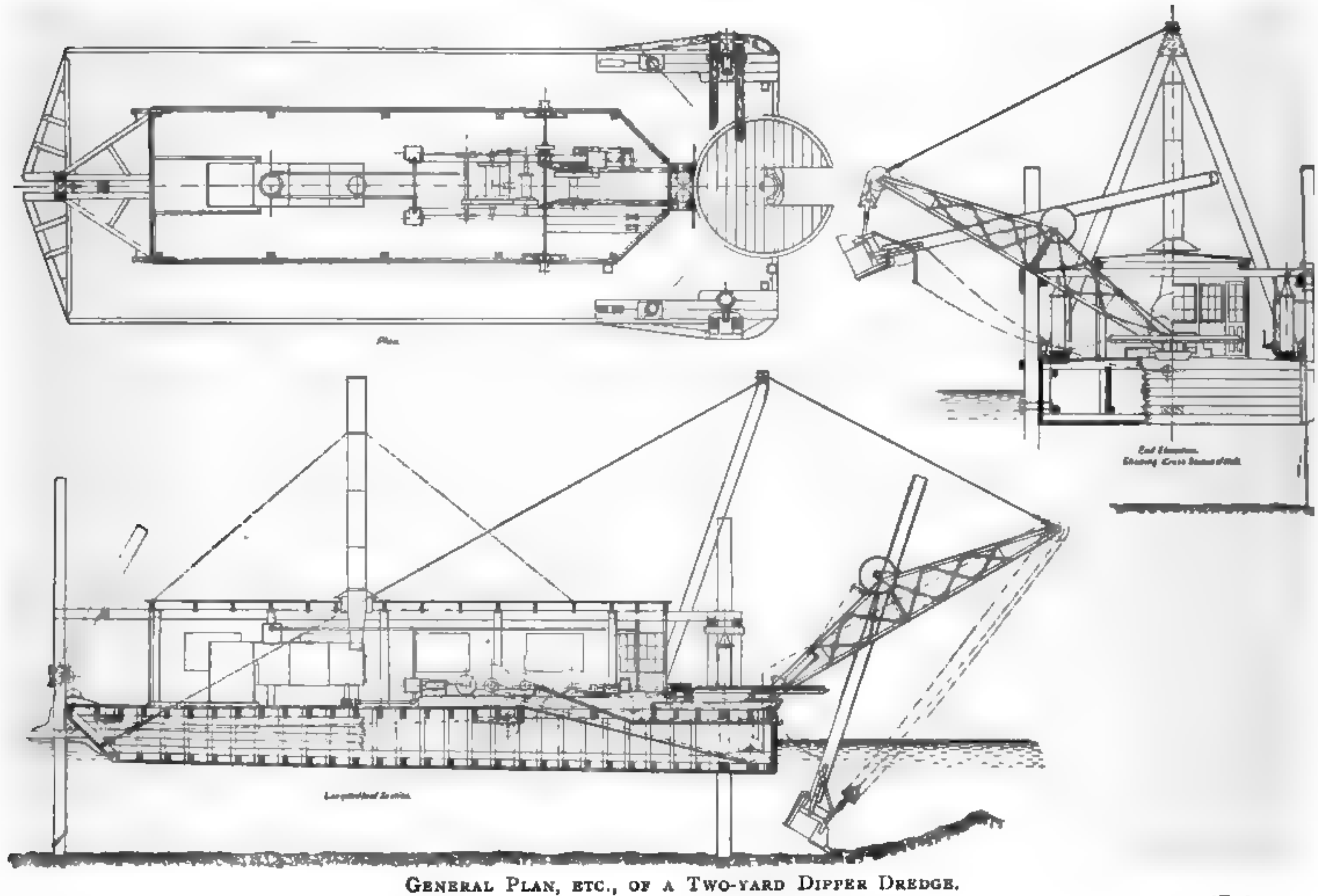
AVERAGE CAPACITY [illegible] CU. YDS. OF ORDINARY RIVER SAND PER HOUR THROUGH [illegible] FT. OF FLOATING PIPE.



(To face p. 48.)



Section of an Elevator Dredge.



General Plan, etc., of a Two-yard Dipper Dredge.

(To face p. 4

The advocates of the method of dynamite, among whom are experienced engineers, claim that the explosion loosens the material for a wide distance, and thus enables the current to remove it. Experiments with eggs buried in the sand showed that one was broken at a distance of 110 feet, while others, from 115 feet to 315 feet away, were unharmed, the charges being, it is believed, about 100 pounds.*

Snagging.—In order to make navigation safe it is frequently necessary to remove such obstructions as snags, rocks, wrecks, etc., and to cut such overhanging trees as may interfere with craft. This work is known as "snagging." Nearly all navigable rivers of importance in this country have steam snag-boats, provided with suitable grappling and lifting appliances, explosives, tools, diving apparatus, etc., and go over the rivers at the most favorable times to remove obstructions. The snags are sometimes cut or sawed into short lengths and placed upon the banks, where they will dry out and float off when a rise appears; sometimes they are boated to deep pools and dropped, sinking to the bottom; and sometimes they are split up by dynamite, dried out, and burned. On rivers of small draught where steam snag-boats could not move about during the low-water season, the work is done with tools and explosives carried on push-boats or bateaux. These boats are usually 10 to 15 feet in width, and 75 to 100 feet in length, and draw but a few inches of water. They are propelled by the crew by the use of poles. As they can move on a small depth of water, the low-water season is selected for the work, and it can then be done very effectively and economically, the snags all being in sight.

On some rivers, as, for example, the Ohio, wrecks of barges are a frequent cause of obstruction. The large tows occasionally become unmanageable and strike bridge piers or other obstacles, and some of the coal boats and barges are sunk. Their speedy removal is frequently necessary, particularly in the upper part of the river where the coal rises pass off quickly. Dynamite is used for this purpose as far as practicable, and sometimes dredging is resorted to. We have seen coal boats removed thus within a few hours after sinking, so that tows could pass without delay or danger.

One of the worst things to be contended with in many American streams is the driftwood, which sometimes accumulates in piles and forms serious obstructions. Tributary streams, where flowing through districts being cleared up or in which logging operations are going on, put out great quantities of tree-tops, logs, trees, and drift of every variety. In addition to the débris which thus finds its way into a river from tributaries, the river itself when in flood undermines its banks, and brings in further débris of similar character. Where these lodge in the bed they become partially covered up and form obstructions to navigation. A bar which in itself is not a dangerous obstruction may thus assist in holding things which are dangerous. In fact one snag rightly lodged may, and frequently does, change the course of the channel very considerably.

* Engineering News.

CHAPTER II.

REGULARIZATION.

Theory.—In a stream in a natural state there is a constant change going on. The banks are being cut away, bars are forming, channels are dividing, and the bed is shifting. In looking for a remedy the idea is naturally suggested of building artificial banks and thus modifying the régime of the river. The contraction thus effected will accelerate the current, and a new and controllable direction will be given to the water, so that it may be expected that a more uniform condition of the stream will be brought about. Thus is conceived the possibility of securing a better waterway than the original, and one which will be exempt from many of the evils existing in rivers in their natural state. In this natural state there exists a series of pools and shoals. In the pools there is ample depth for navigation, but at the shoals the water spreads out into a superficial sheet, and to secure and maintain a navigable depth this must be confined to narrower limits by structures built in the bed of the stream.

General.—Regularization, or regulation as it is sometimes called, has been applied in Germany on a very extensive scale, and has been employed in all countries to a considerable extent. It has for its object the establishment of a bed for ordinary and low-water stages, and the equalization of the slope. To this is sometimes added the duty of protecting the banks above these stages. Its comparatively low cost and the rapidity with which the works can be put into execution have recommended it for rivers of great length and fall, where canalization would have been out of the question on account of the expense and the length of time required to construct the works, so that hundreds of miles of river have been rendered navigable where, without this system, commerce would now be impossible, or, at least, very uncertain. In many cases, in fact in the majority in this country, the work of regularizing is incomplete, having been applied only at those points giving most serious trouble to navigation. The conditions originally existing have been greatly improved by a general cleaning of the channel, followed by works of contraction and correction here and there, having in view the concentration of the river at low stages to a single bed, the deepening of the water, and its guidance in more uniform widths; and the banks have been protected at many of the places where most seriously menaced. However, very little of this work is complete, and it will require large additions and extensions before the desired results are obtained. In fact much of it has been experimental in character, and many of the failures have been due to the lack of an extended study of the existing

conditions when planning and locating the work. It is not an uncommon thing to see in some of the earlier works of regulation examples where injury instead of benefit has been done to navigation by their faulty location. Another frequent cause of failure has been the lack of sufficient funds with which to properly perform the work.

Numerous failures have naturally led to bringing the system into disrepute, but it is believed that much good can be done by it where the plans for its execution have been well studied and are based on correct hydraulic principles, and the necessary time taken in which to carry them out. There will be opposition from navigation interests, which naturally demand immediate relief, but permanent results are more advantageous in the long run.

As has been stated, regularization contemplates the establishment and maintenance of a medium and low-water channel. It is important that this channel shall have sufficient depth during the lowest water liable to occur, as well as at ordinary stages, and be of sufficient width to accommodate all navigation. Should it fail in depth, craft must proceed with lighter loads, or stop altogether. The desired depth is obtained by decreasing the section, that is, by narrowing the channel at bars, etc., by artificial works. This contraction of the river-bed increases the current velocity and causes erosion, the material being carried out into deeper water below. This very contraction, however, may defeat the ends of navigation by leaving an insufficient width for navigation, and by increasing the current beyond the limit at which craft can proceed safely and economically.

The following rates of current for different rivers are given by Sganzin, in his "Cours de Construction":

	Feet per Second.
Mean velocity of the Seine, below Paris	2.3
" " " " Thames at London, flood tide	3.0
Low-water velocity of the Tiber at Rome	3.3
" " " " Danube at Ebersdorf	3.5
" " " " Loire	4.3
" " " " Rhone at Arles	4.9
" " " " " " Beaucaire	8.5
" " " " Durance, below Sisteron	8.5
" " " " Maragnon, S. America	13.0

Velocity of the Rhine varies from 3 feet 2 inches to about 14.0 feet.

In Europe, where a depth of three feet is considered sufficient to permit a traffic with barges or canal-boats, it has been found that where the slope of the river exceeds 1 in 2000, up-stream navigation becomes very difficult for the ordinary methods of towing. On the river Lys, in Belgium, for example, where the slope is 1 in 2000, the towing is done by horses, and would be very arduous were it not for the presence of the aquatic plants which retard the velocity of the current, and which it is strictly

forbidden to cut.* On the Rhone, where the slope is nearly 1.6 in 2000, steamboats of special power are needed to ascend the current.

The bars to be acted upon by the current usually appear at the wider places in a stream, and are composed of sand or fine gravel, so that an increase of velocity, even in a slight degree, will set some of the lighter portions in motion. If it be greatly increased the bed may be scoured out beyond the depth desired and thus lower the water-surface at points up stream, and expose new obstacles to navigation.

In rivers with unstable beds the fixing of a permanent passage is difficult and expensive, but when the bottom is of hard material there is usually no reason why a fairly good channel may not be established, and in such cases regularization would seem a very economical method of improving a river were it only necessary to contract the water-way at each offending bar.

Applicability.—The artificial constructions thus indicated are, however, usually not all that is necessary to satisfy the requirements of navigation. The system has its drawbacks as well as its advantages and it is not practicable to apply it in all streams with success. Prof. Engels, a German authority, thus sums up his conclusions in regard to it: †

"(1) Only rivers, or long reaches of rivers, in which natural erosion is fully developed are adapted to regulation. The navigability of unfinished rivers, still in a state of erosion, can be improved with permanent results only by canalization.

"(2) The most that can be accomplished by regulation is the desired adjustment of the slope of the low-water line, and this only on reaches of uniform regimen and uniform characteristics.

"(3) This adjustment of the slope, to be accomplished when the conditions are most favorable, can only be established and brought about by constructive measures after the formation of that part of the channel which rises above low water is completed; that is, after the conditions of the bed have adapted themselves to the change of energy caused by the formation of the mean high-water bed—in other words, after the erosion caused by this formation has come to rest.

"(4) To secure the establishment and permanent preservation of the adjustment of slope, the irregularities of the bed in the longitudinal and transverse profiles are to be adjusted after reinforcing the low-water shore, and the bed is to be strengthened where attacked by the water on account of the ground plan of the channel. Restriction of width alone will not bring about that degree of navigability which may be desired."

Hagen, the eminent German engineer, in his treatise on river engineering, thus speaks of the value of regularization:

"In no case will regularization allow the full attainment of the proposed result. The result obtained depends upon the special characteristics of the river in question, and in particular upon the magnitude of discharge and the fall, so that it is usually

* "Civil Engineering," Law and Burnell. † Transactions Am. Soc. C. E., vol. xxix, p. 220.

impossible to exceed a certain degree of improvement, usually very limited. It must be remembered, where the regularization has in view the discharge of flood waters, that the reduction of level depends on the amount of fall, and if the latter is considerable, the reduction of level and the effect on the discharge of floods will also be considerable; but quite different is the aim of regularization when it is a question of improving the conditions of navigation. We have then to obtain a sufficient channel depth in times of low water, and so called regularization works are in themselves usually powerless to give a suitable depth unless the water-level is raised by means of dams or other similar constructions."

Notwithstanding these opinions there are numerous examples of fairly successful improvements of this character on the Garonne, the Rhone, the Rhine, the Volga, the Elbe, and other streams, and it is fair to conclude that when the river-bed is of a stable nature and the discharge considerable, regulating works will be successful if the plans are made with a view to the general rather than the local effect, and are designed on correct principles and without a too great regard for economy.

The greater number of regularization works in rivers rest on a purely theoretical conception, the end to be attained being to realize in a natural stream with a bed more or less movable a uniformity of slope and regularity of section. Had these streams been cut through material capable of wholly resisting the action of the current the conditions existing on artificial canals would have been found, and success would have followed intelligent application. With the permeable material through which nearly all rivers flow, the object attempted has rarely been realized.

Some of the works of contraction, however, have been founded on observations of natural phenomena over a long period of time, supplemented by patient and intelligent experimental researches, and these have had a fair measure of success, although not always meeting the full requirements of navigation.

It will be interesting to summarize the results in a case of this kind. The example is that of the Garonne and the notes are made from a paper by Inspector-General Fargue,* under whose direction the works were executed. The general régime of the flow, following the laws which govern all rivers, was as follows:

The center line or channel followed the concave bank.

The bars were deposited along the convex bank.

The channel was the deeper and the bar the more projecting as the concave or convex curve was the more accentuated.

The maximum or minimum of curvature corresponded respectively to the maximum and minimum of depth. This correspondence did not occur in the same transverse profile, the deep place being below the concave summit, and the greatest projection of the bar below the convex summit.

The least depth was below the points where the concavity was changed to convexity. The channel presented regularity in its longitudinal profile when the

* "Rational Direction of Artificial Banks," etc.

curvature of the axis of the bed varied in a gradual and continuous manner, and every abrupt change of curvature was accompanied by an abrupt change of depth.

These relations existed only in those portions of the river where the length of sinuosities, that is, the distance between two consecutive points of inflection, was neither too great nor too small. Where these relations did not exist, the channel was formed of isolated trenches separated one from the other by shoals or bars which soon reformed after they had been removed by dredging.

Using these facts as a basis on which to reason, M. Fargue formulated the following rules:

1st. In order that the channel may be stable and permanent, it is necessary that each bank present a succession of curvilinear arcs, alternately concave and convex, and connecting right lines formed by the prolonged direction of parts of the banks where the curvature changes directions.

2d. In order that the channel may be deep, it is necessary that the polygon formed by these right alignments have angles and sides neither too great nor too small.

3d. In order that the channel may be regular it is necessary that the curvilinear arc have gradual curves, that is, approaching a curve whose curvature is nothing at its inflection, grows in a continuous manner up to a certain maximum, and decreases afterward to again become nothing at the following inflection.

4th. The spacing of the banks should vary with two elements, the distance and the curvature, to wit: on the one hand, the width at the point of inflection should increase going down stream, and on the other hand, between two consecutive points of inflection the width should grow with the curvature and present toward the apex a maximum which is the greater as the curvature of the apex is greater. The width increases then, according to a periodical law, in such a manner that the bed is found widened toward the apex of the curves and restricted in the region where the curvature changes its direction.

5th. In this same region the points of inflection of the two banks should not be in the same transverse profile. The one where the concavity is changed to convexity should be above that where the inverse change is made, at a distance which seems to depend only on the width at the point of inflection.

In the note from which the foregoing extracts have been made it is stated by M. Fargue that the pass above Caudrot has presented constantly for forty years depths varying from 10.7 to 13.12 feet; that of Mondiet, which between 1852 and 1866 had only 30 inches depth, in 1886 had 6.56 feet; part of this increase was accomplished by dredging, but it was accompanied by a correction of the outline of the left bank under the rules just laid down. With a single dredging in twenty years the depth has remained between 4.3 and 6.6 feet. In another pass the depth has been increased by this process and maintained constantly.

There is one fact in connection with these improvements worthy of note. In that portion of the river having these contracting works the bed has eroded at the upper

end and filled in at the lower end; the mean slope has therefore become less. This seems to be a certain result of contraction, no matter under what theories the works themselves are planned. In regard to this De Mas says:* "In a stream with a movable bottom, a continuous diking, diminishing the width of the section, produces the following effects: Toward the upper end a lowering of the bed takes place, and this has as a consequence a corresponding lowering of the bottom of the plane of the water. Toward the lower end there occurs a raising of the bottom of the plane of the water. In all the diked part a diminution of the mean slope of the bottom and surface takes place."

As has already been stated, regularization works have been constructed on a large scale in Germany, and the result of these works has been an enormous development of navigation on the principal rivers of the empire. On the Rhine the development of such works began about three-quarters of a century ago. The system adopted was that of spur-dikes placed close together, longitudinal dikes, parallel dikes, dredging in the river-bed, and bank protection. Similar works, but with much less development, have been established on the Elbe and Oder, and on other rivers.

In the beginning the realization of a normal profile and a uniform slope were aimed at, but later studies and experience produced an evolution in ideas. Prof. Schlichting has laid down the following principles upon which rests the German system of regularization:

1st. Construction at the convex banks of spur-dikes, and in the concave curves of protection dikes and dikes parallel to the banks, with the use of inclined submerged spurs to close unusual depths.

2d. Transformation of slight sinuosities into right lines, adopting the best possible location of the channel in the navigable bed.

3d. Systematic alternation corresponding to the alternation of convexities and concavities, the spur-dikes being on one bank with works of protection and dikes on the other.

4th. Contraction of the width of the river by the advancing into the bed of convex curves, accompanied by a small flattening of concave curves.

5th. Diminution of the mass of material transported in the navigable channel, to be accomplished by the consolidation of banks of alluvium in the convex parts.

In regard to these works it is stated on good authority that "the incontestable fact is that by adopting a process of exploitation appropriate to the navigation, the Germans have succeeded in serving an enormous traffic with relatively small depths; and on the other hand it is certain that the works of regularization built have been in general relatively of small expense."

It would seem from the foregoing that it is possible to deduce certain general conclusions upon the regularization of streams, and De Mas † undertakes this as follows:

* "Rivières à Courant Libre," p. 325. † Ibid.

"Let us consider a navigable stream, or rather a section of this stream comprised between two important affluents. The elements of the régime of this section of the stream are:

"1st. The total slope of the valley from one extremity of the section to the other and the total development of the stream between the same points which determine its mean slope.

"2d. The nature of the materials of the bed and banks, and the nature of the material brought from the upper part of the river or from its affluents.

"3d. The discharge in low water and in floods, and the slow or torrential character of the latter, etc.

"From these different constitutive elements the results affecting navigation are the trace of the center line, the width of channel, the depth, and the velocity of current. For the section of stream considered there exists one combination of these results, which is the most favorable from the point of view of navigation, a combination which nature does not generally realize spontaneously, but which works of regularization judiciously conducted permit us to obtain. It is only long experience based on patient observation, and frequently on repeated defeats, that will enable this problem to be solved. It cannot be told *a priori*, and this explains the diversity that exists in the depths chosen for the different rivers of Germany. There has been adopted for each the greatest depth in low water compatible with other conditions more indispensable. In fact if the navigation should be certain of development on a river (in the contrary case it is best not to be concerned with it) it can only be conducted with facility and security on the condition of having a sufficiently wide channel. When we observed the contractions that have been attempted on the Meuse, we found 35 feet to 39 feet adopted for the width of the normal profile at the bottom. Such a width may suffice to permit the passing of two boats in a canal, but it is incompatible with navigation of any importance in a river. A width of channel from 75 feet to 90 feet should be considered as the minimum. On the Rhine the channel is 450 feet in width.

"On the other hand the velocity of the current must not exceed certain limits. A rapid current may be an obstacle to economical traction in ascending, and consequently to the development of navigation.

"We may therefore conclude that works of regularization will only give satisfactory results on streams of large discharge and generally of moderate slope. These two conditions are most often found united in the régime of the rivers of Germany, and it is that which explains the success of works of regulating in that country. They are found also more frequently in the middle or lower parts of streams than in their upper parts; so that works of regularization may be justified for certain sections of some streams and not for others.

"If the bed of a river is composed of drifting material on which the water can act it naturally results that the depth obtained by diminishing the width of the section carries with it the lowering of the former level of the river above the point considered.

The results of all works of regularization have sufficiently demonstrated this. The slope of a river, whatever it may be, is never the same in all parts of its course; its low-water surface, in particular, forms a broken line with the least slope in the deep pools, and a much greater fall where there are bars. In lowering the level above the bars new obstructions are developed, shoals which were unnoticed before, as they were hidden under an ample depth of water. This, however, is not the result sought, and to counteract such evils it is necessary to reduce the section of flow of the river above the bar, and at the same time prevent the current from digging the new channel too deep. For this latter purpose the bottom must be covered with fascines, riprap, etc.

"Besides the fact that new bars are brought to light in the upper pool, daily experience shows that the alluvial matter which the current takes away from the contracted section goes to form new shoals farther down stream. Thus the regularization of a river, when only attempted at places where there are bars, does not ordinarily give the desired results. After a certain time it becomes absolutely necessary to extend the contracting dikes for the whole length of the river. We have numerous examples of this in the rivers of western Europe, which have been subjected to works of systematic regularization. Upon nearly all we find a continuous protection of the banks for their whole length, and works designed to narrow the river-bed follow one another without interruption.

"In all cases the improvements that may be expected from these works cannot pass certain limits which it is very difficult, not to say impossible, to predetermine; the results must therefore be unknown.

"It may also be asked if the form of the regularized bed established in low water will resist great floods. It seems certain that works of this nature can be preserved, but only by a vigilant maintenance. This maintenance is, moreover, the more necessary and expensive as the works are of more recent construction or isolated, and not yet consolidated into a system of complete regularization. Serious injuries happening to these works may result in changes in the channel, especially where it is naturally of a shifting character. It is only just, however, to observe that there are no works which do not require repair, and that are not subject to injuries affecting more or less widely the régime of the lines of communication upon which they are established.

"While the authors of the first projects followed a uniformity of method which is not compatible with the natural régime of streams (their object being to secure a uniform normal profile, and uniformity of slope), the aim to-day in works of regularization is to obtain a continuity as perfect as possible in the longitudinal and transverse profiles as well as in the outline of the banks and of the center line. Every abrupt change in the outline of the bed is considered as forming an obstacle to navigation. . . .

"Submerged spur-dikes may often be employed with success in preventing or correcting the inconveniences resulting from the construction of a bridge. Thus, on the Saône at Lyons, the bridge of Ainay, now under reconstruction, the arches of which

were narrow and incumbered with off-sets of masonry, never afforded in high water more than one arch practicable for navigation, and that one could be used only with difficulty. The later construction of submerged sills below sufficiently distributed the fall so that the same boats were able without serious difficulty to pass the arches which before they had not been able to approach."

CHAPTER III.

DIKES AND THEIR EFFECTS.

THE structures for guiding a stream along a caving bank and protecting the same from undermining, for confining and directing the water at bars and shoals, and for closing secondary arms of a river, are all known under the general name of Dikes.*

Types.—There are three general types of dikes in use in this country and abroad called from their location and construction, spur-dikes, longitudinal dikes, and submerged spurs or "Grundschwellen," which have been used to a considerable extent in Europe.

The two forms of spur and longitudinal dikes are not infrequently combined. When this arrangement is adopted it is known as the mixed system.

Spur-dikes.—This system has had an extended application in Germany, on the Rhine, Elbe, Vistula, Oder, and other navigable streams, and has been employed in several other countries, including the United States. It attempts to perform its mission by means of dikes placed at intervals along the shores, and projecting more or less into the stream (either normal to the channel or slightly inclined down stream), and partially closing its low-water bed, instead of lying parallel with it, as in the longitudinal dikes. The chief object of these works is the improvement of navigation, but the deposit of alluvium between the various spurs where they have been in existence for a long time may become a matter of considerable importance; and is in fact one of their most useful and most valuable features. In the course of time these deposits reach the level of the dikes and solidify them and protect them from ice and floods. A new and continuous bank uniting the heads of the dikes is thus formed. It is stated that 8400 acres of mean river-bed on the Prussian part of the Rhine has thus been transformed into alluvial soil. This class of dike at first decreases the flowing section of the river at the head or outer extremity only, so that it is evident that there must be more than one, in fact a system, in order to effect a longitudinal deepening across a bar. The bank also may be cut away opposite these various spurs, in which case recourse must be had to another system for protection, built upon the opposite shore.

Mention of the spur dikes used in America will be found in the chapter on "Bank Protection."

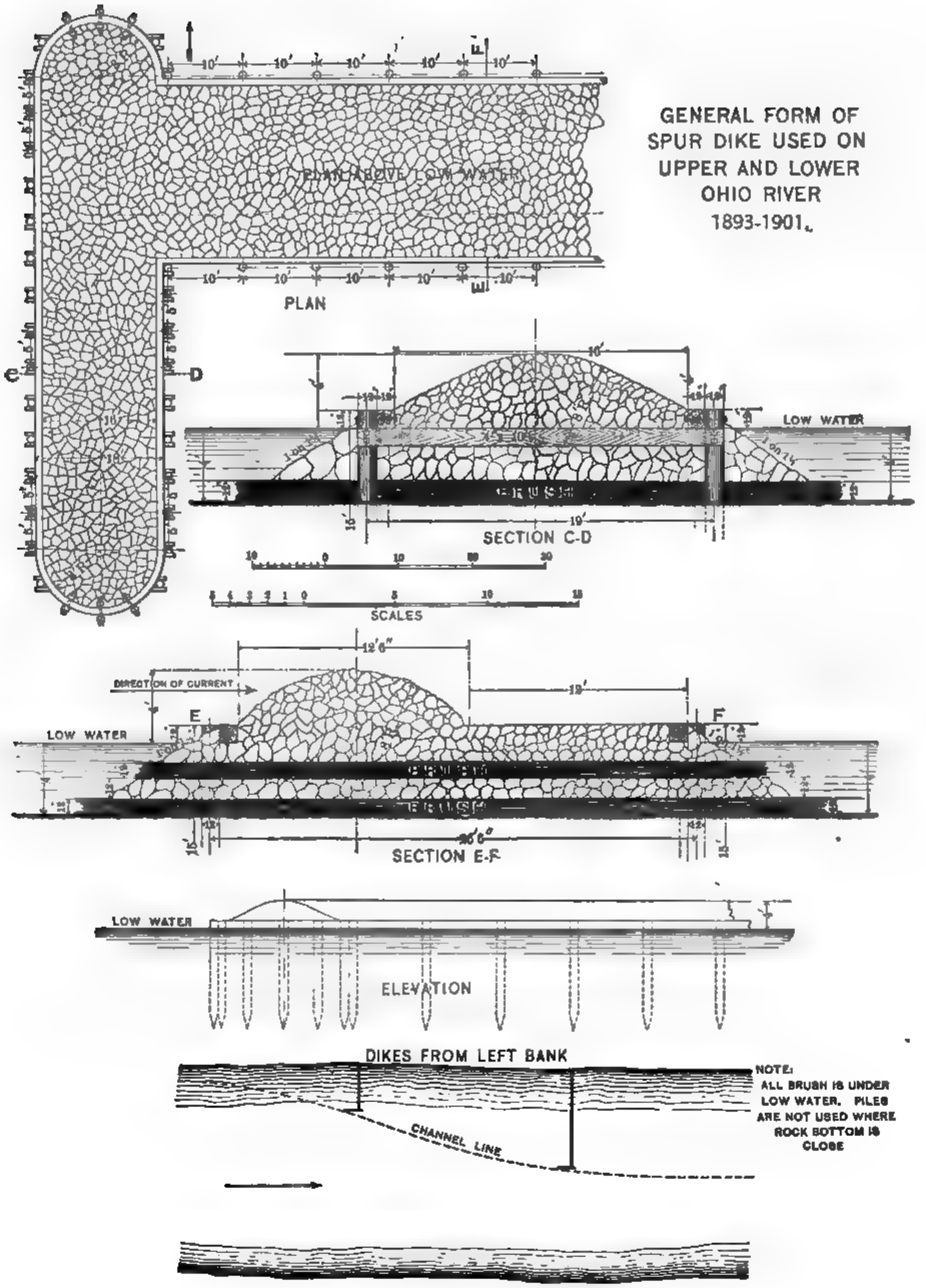
Longitudinal Dikes are built along the bank about parallel to the direction of the current. Their use has been very extensive in Europe and they have met with consider-

* Some of the plates accompanying this and the next chapter are republished by permission from "The United States' Public Works' Guide and Register," by Captain W. M. Black, Corps of Engineers, U. S. A., and from the papers by H. St. L. Coppée on "Bank Revetment," published in Transactions American Society of Civil Engineers, January, 1896.

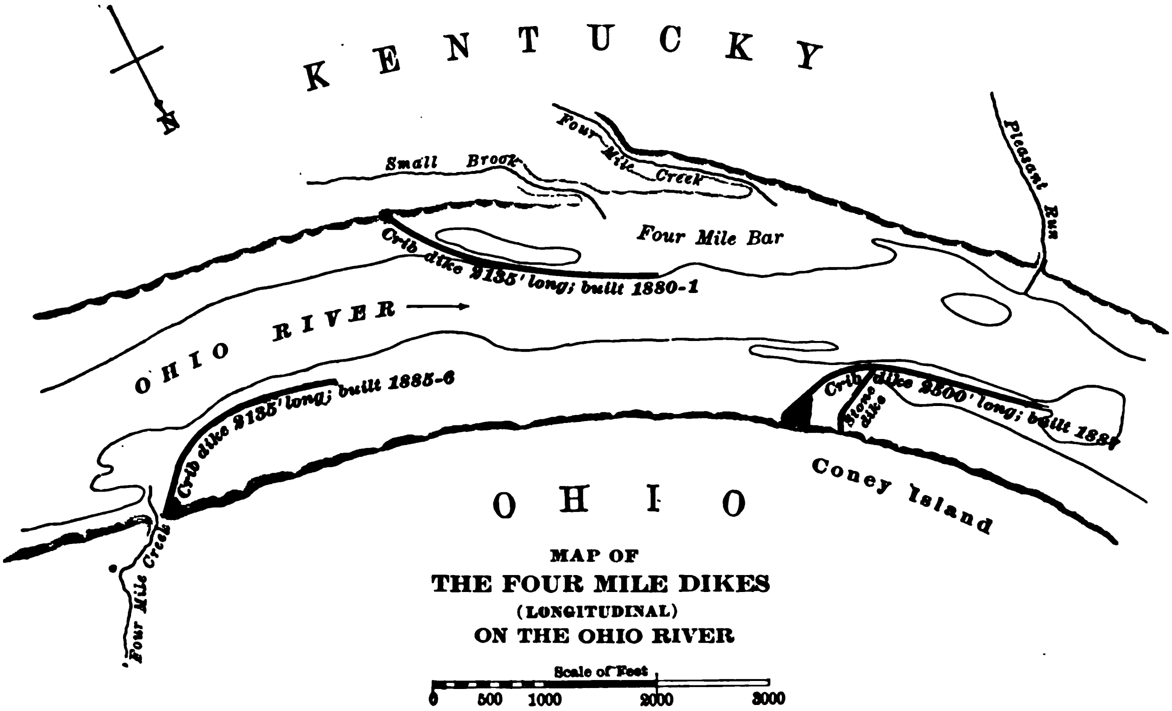
able favor in this country. Their object is the same as that of spurs, but they accomplish it in a way which, while probably more certain, is sometimes more expensive. Those along the Ohio River are frequently built of wooden cribs filled with broken stone. Starting from the bank they describe a regular curve down stream until their direction becomes parallel to that of the proposed channel, when they follow the line of the latter. The space behind some of them gradually fills with sediment and even grows up with willows, and a new bank is formed, while behind others considerable scour takes place. They are effective in scouring out the bars and affording a greater depth at low-water seasons. At ordinary stages their crests are submerged so that boats may pass over them in safety. Their position is such that the dikes themselves form the banks of a new, contracted channel with a more regular current than found when spurs are employed; but, on the other hand, longitudinal dikes for their whole length are subjected to the continual action of a current swifter than that which previously existed, and under these circumstances the dikes may be undermined, and even destroyed. The space included between a longitudinal dike and the bank which it is designed to replace is not immediately filled up by alluvial deposits, so that when a rise comes the water overflows the dike, and, pouring into the space behind it with a velocity due to the height of fall, tends to undermine the dike on the inner side. Their maintenance requires constant watching, and the expense of repairs is considerable. Negligence in keeping them up may be the cause of disturbances over the whole area included between the two dikes, and this would happen more rapidly than with spur-dikes. Lastly, the use of longitudinal dikes does not allow, except at considerable expense, subsequent contraction or enlargement, if either should become necessary.

Submerged Spurs.—We have seen that one of the effects of contracting a river is the scouring out of its bed. This is, of course, one of the objects aimed at, but this action will not always cease at the point desired, and hence it sometimes becomes a menace rather than an aid to navigation. To correct this evil it was proposed many years ago to adopt in connection with the dikes a system of low dams or sills placed at intervals and dividing the river into a number of sections. They would rise approximately to the river-bed and in reality form immovable bars. This conception has not, so far as we know, been exactly applied, but the same general idea has received a considerable application on the Elbe, Rhine, Rhone, and other rivers. The works are called submerged spurs, ground-sills, or "Grundschwellen." It will be noted that they are not considered as a complete system of improvement, but are employed in connection with regulating and contracting dikes. Their chief object is to raise the bottom and induce deposits, the minor bed of the stream being at the same time held to its place by supplementary works at such places as desired.

Generally speaking the term ground-sill is applied to works constructed under water, which stop at a depth somewhat greater than the normal depth of the stream, and which are built in order to strengthen and consolidate the bottom of the


GENERAL FORM OF SPUR DIKE USED ON UPPER AND LOWER OHIO RIVER 1893-1901.
PLAN ABOVE LOW WATER
PLAN
LOW WATER
BRUSH
SECTION C-D
SCALES
DIRECTION OF CURRENT
SECTION E-F
ELEVATION
DIKES FROM LEFT BANK
CHANNEL LINE
NOTE:
ALL BRUSH IS UNDER LOW WATER. PILES ARE NOT USED WHERE ROCK BOTTOM IS CLOSE

(To face p. 110.)



MAP OF
THE FOUR MILE DIKES
(LONGITUDINAL)
ON THE OHIO RIVER

(To face p. 61.)

river-bed, or else with a view to raising it at points where too great depths exist. They are built in nearly the same way as the spur-dikes which rise above the water-surface, and of the same kind of material, and usually start from low-water mark, inclining in direction up stream at an angle varying between 60° and 80°. The top slope as it goes out into the river is about 1 in 10 to 15 for a short distance and then decreases considerably until it connects with the opposite shore.

The earliest example of submerged spurs is found on the Ruhr, a little river which falls into the Rhine at Ruhrort, and to which the coal mines of Westphalia gave an unusual importance in the system of transportation routes prior to the construction of railroads. The Ruhr was made navigable for a length of 46½ miles by a canalization comprising eleven locks and dams. It happened that one of these pools was scoured out, and the slope in low water disappeared, so that the miter-sill of the lock at the head of the pool was uncovered, and navigation was stopped. The engineers restored the slope by constructing throughout the pool a series of submerged transverse dikes, which, even before the filling up of the bottom, divided the total fall so as to restore the depths necessary for navigation during low water.

Great use was made of submerged spurs in the improvement of the Elbe, and it may be said that the German engineers have been naturally led to them by the system of works which they have adopted. They have aimed at improving the channel by contractions, by creating in the natural bed a minor bed whose width, after taking into account the degree of resistance of the bottom, is regulated in accordance with the conditions of slope and of discharge. Instead, however, of controlling this minor bed by longitudinal dikes, they have created it by building spurs which extend into the stream from each bank with a slight up-stream inclination, and terminate on the line adopted for the desired bank of the minor bed. As might have been expected, the heads of the spurs were usually attacked by the current, and scour produced, threatening the existence of the spurs, and destroying the regularity of the channel. The engineers were thus led to prolong their spurs under water, advancing into the bed of the river in order to protect them and limit the effect of the scour.

The works of regulation on the Elbe are as follows:

First: next to the bank is the spur-dike intended to contract the natural bed. Usually this dike at its root on the bank is at a height of 8½ feet above low water, and at its outer end at a height of 6¼ feet above low water.

Second, in prolongation of this is the submerged spur or sill which limits the scour caused by the dike proper and protects its head. The starting-point of the sill is about 5 feet below low water, and the top has a slope which may be from 1 foot in 25 to 1 foot in 12.

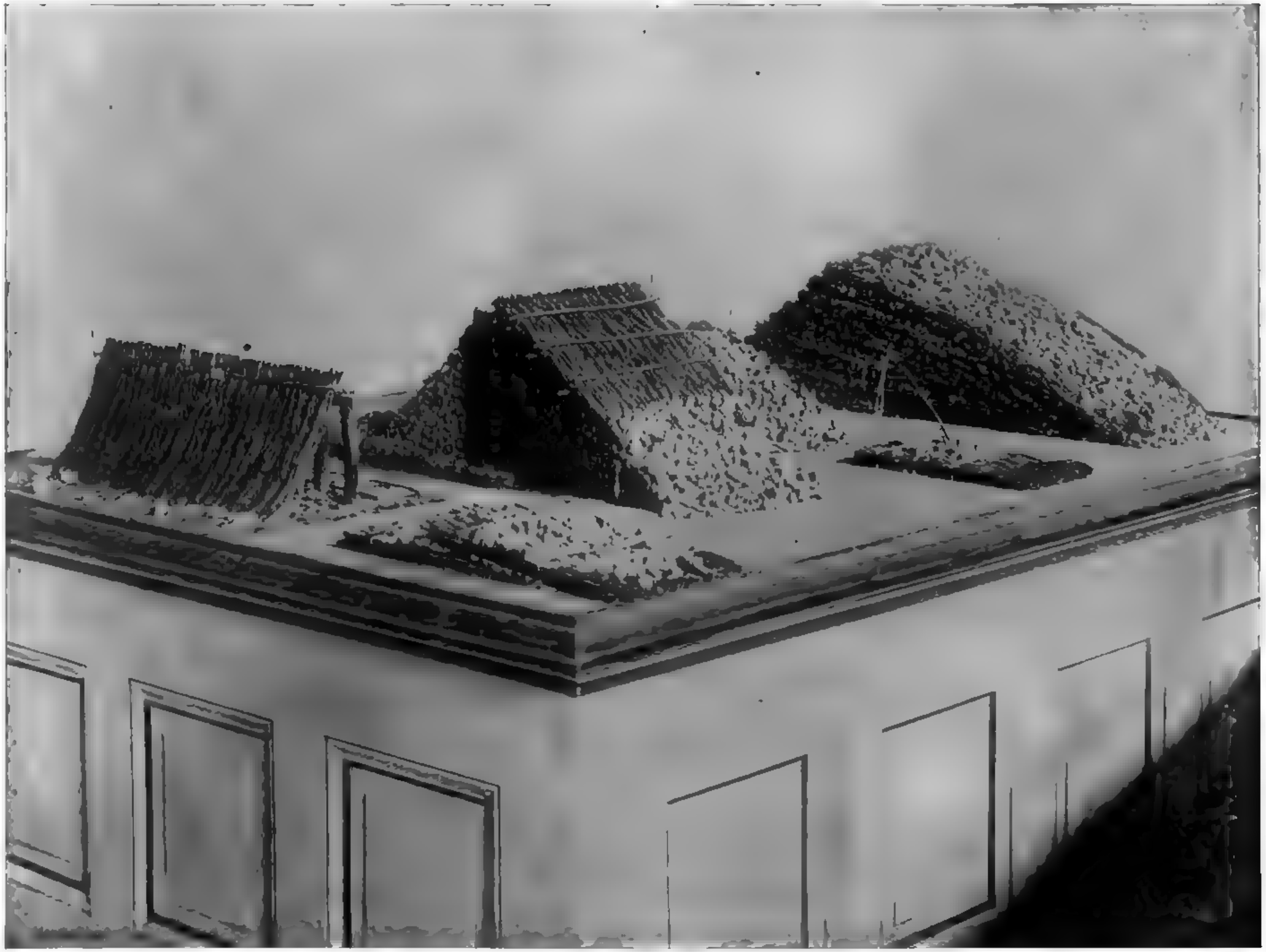
The works thus placed have completely answered the expectation of the engineers. The alluvial deposit has filled up the places that had been scoured out, and has permitted a reclamation of the spaces between the spurs. The works have thus been secured definitely, and other results have been produced which are still more important

for navigation, in diverting the principal current from the heads of the spurs, and pushing it forward into the open bed to the line of the greatest depth. Barges and rafts floating with the current, and fleets of boats in tow have ceased to be carried against the heads of these spurs, and are now kept by the natural forces in the middle of the channel, or at least at a sufficient distance from the banks. The submerged sills have thus caused the disappearance of one of the inconveniences which could be urged against the system of contraction by spurs—that of forming obstructions dangerous to navigation. The improvement was so marked that sharp bends, whose rectification was formerly demanded, are now free from danger, and they form part of the plans adopted for the improvement of the river.

The sill which we have described is usually somewhat short, but in places where considerable scour has been produced, or is to be feared, they no longer have as their object the protection of the spur-dikes, but are constructed with a view to the regularization of the bottom, and consequently of the slope. Hence the works on the Elbe have in a great measure made the depths uniform. Formerly this river, as in most streams, had a series of pools, more or less deep, separated by rapids. To-day the bottom has a nearly uniform depth. It is true that the velocity of the current has been increased in the parts corresponding to these pools, but the advantage of a regular depth of water over the whole route, and the other advantages which result from it, are such that even this inconvenience is not of great consequence.

Submerged sills, which may render such considerable service as permanent works, are not less useful as a preparation for other works, and for their economical execution. The work of regularization by spur-dikes is not carried out immediately, as is the case when longitudinal dikes are built. Their construction in Germany is accompanied by methods and rules which permit a considerable liberty of action on the part of the engineer. Hence, when an improvement is liable to lead to a great displacement of the bed, especially in the concave parts of a stream, the dikes are commenced on a short length of river, and stopped at a provisional curve. Time is then taken to watch the effect produced. If the action of the current attacks the bottom at the outer ends of the spurs, they are prolonged by sills which form a foundation for the succeeding part of the dike, and which in the meantime immediately stop the scour and cause deposits. This gradual construction, or experimenting with the dikes, is done not only as regards their width, but also as regards their height, and when a certain depth has been obtained, submerged works are commenced, which at a later period, are built higher in proportion as the expected effect is produced. Remarkable effects of deposits are thus obtained, and dikes which could only have been constructed at great cost in deep water, are built gradually, and are finished on a bottom that has been raised without difficulty and at a small cost.

On convex curves the results are very rapid, but on concave curves the system of spur-dikes gives less satisfactory results. Scour at the heads of the dikes always takes place, and it cannot be checked except by the precautions and the gradual



MODELS OF BRUSH AND ROCK DAMS AND DIKES. IMPROVEMENT OF THE UPPER MISSISSIPPI RIVER.

(To face p. 6[illegible])

method of construction just described. It was doubtless this fact which induced the German engineers to adopt and generalize in so remarkable a manner the use of submerged spurs, the other advantages of which could not have been discovered except by the experience acquired after their construction. In fact, in all that portion of the Elbe which is under the control of Prussian engineers, and where they have applied sills, the two banks to-day show a remarkable regularization, and one which constantly improves, so that the indentations still visible between the successive dikes are filling up and new and regular banks will before long be in existence.

It will be seen from what has been said that regularization of the bed as well as of the banks is considered necessary in Germany, and that they not only fix by a series of dikes the width of the low-water bed but also secure this channel against scour by means of transverse dikes. The result is that the bed rises to the level of these sills and assumes a regular slope, just as the banks become regular by deposit between the spur-dikes.

The advantages of this system are summed up by M. Jacquet as follows:

The nearly uniform distribution of the slope, and the consequent disappearance of the bars over which the depth of water was not in harmony with the general regimen of the river.

The protection of the works of regularization, and in general of all the works that were attacked by shore currents.

The removal or transfer of the line of greatest depth and greatest velocity to a certain distance in front of the shore works, and the consequent suppression of the dangers which dikes might offer to descending navigation.

The regularization of the depths and of the velocities in the same cross-section.

The creation of a uniform depth of channel throughout the length of river subject to the same regimen, and sometimes throughout the whole course of a river, as has happened on the Elbe, so that boats can everywhere find nearly the same depth of water.

Materials.—As a general thing cheaper materials are employed for dike construction than for works of canalization. To be satisfactory the dikes must concentrate the water at all points where the river spreads out, and it is evident that, in order that the entire cost of the works may not be out of proportion to the benefits expected, the expense per lineal foot must be kept down. Nor is it necessary that the construction should be of great excellence, because dikes may be more or less permeable, their chief purpose being that of giving depth to the river and direction to its current. Frequently they are constructed of loose stone piled up in ridges, or of gravel protected by a broken-stone covering, or of timber cribs filled with stone. That part of the timber under low-water level may be considered fairly permanent, but the portion above soon decays, and suffers from the attacks of ice and the current, and, if not promptly repaired, the dike itself is endangered. Piles are also used, driven into the earth and surrounded by stone, while mattresses of willows weighted with stone,

and brush of all sorts are utilized for dike-building and bank protection. In some localities wood or brush is formed into gabions or baskets which are filled with heavy material, such as gravel or stone, or an envelope of poles is bound together and similarly filled. Dikes are also built of solid stone masonry on some of the rivers of Europe, and concrete has been used for similar purposes in America.

Sections.—The front and back faces and also the coping of a dike should be so constructed as to withstand the erosive action of the current. A simple ridge of broken stone, wide at the bottom and decreasing in width as it rises, with its top paved with stone of large size and irregular shape, is widely used. In another form, when the water is not deep, the river-bed is excavated to the desired depth in two parallel trenches, and these are filled with broken stone carried up to or above low-water level. This leaves a core of river-bed material between the two walls, and this is paved with heavy stone as are also the walls themselves. This paving is sometimes made in the form of a curve rising some distance above low water. Where the water has considerable depth the two parallel ridges of stone are placed on the natural bed so that their inner slopes will meet at the bottom. The V-shaped space between them is filled with a cheaper material, such as gravel. On top of this, as a base, additional stone is placed to bring the structure to the proper height.

There are several forms of dikes composed of timber and stone, in addition to the crib-dike heretofore mentioned. Sometimes a single row, sometimes a double row of piles is driven into the bed and the piles connected by wales, and stone is piled around them, generally up to low water. The portion of the dike standing above low water is then planked up. When there is a considerable tendency to undermining, which takes place more or less along the faces of all dikes, and especially where their orientation approaches a direction normal to the current, sheet-piles are frequently used, and these are also protected below the water-line by broken stone.

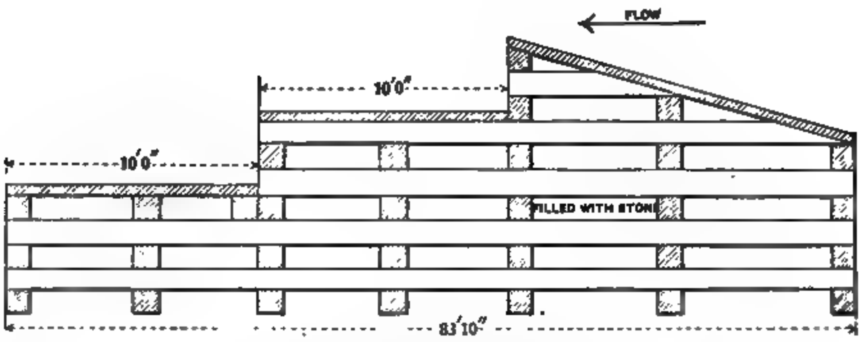
Construction.—The general process of constructing broken-stone dikes is to dump the stones loosely into the water from barges, the line of the dike being indicated by piles driven at intervals into the river-bed. These piles serve also to afford convenient moorings for the barges. The stones are left with a natural slope. When the river-bed is soft the action of the current will cut part of it away, causing the riprap to sink. In the course of time, however, it will become stable and the dike will take a curvilinear form with a wide base. To prevent any after-movement the foundation is sometimes prepared in advance of the construction by dredging out the material to a desired depth.

The settlement which takes place renders it advisable that the coping be loosely put down, so that it may accommodate itself to the movements of the dike. If it is closely placed and the dike should sink, the paving may fall in irregular piles leaving portions of the dike exposed to the current where they cannot be seen and repaired until the low-water season comes. This coping or paving should be of very heavy stones, not only in order to withstand the force of the current but also to successfully resist the pressure from ice-gorges. The main body of the dike may be made up of

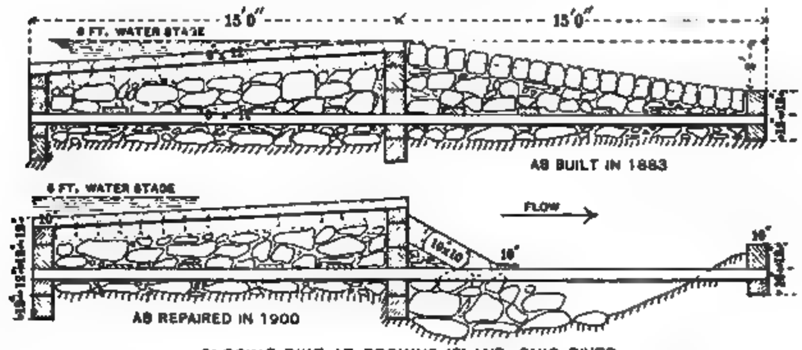
SECTIONS OF
CLOSING AND LONGITUDINAL DIKES
ON THE OHIO RIVER.

SCALE OF FEET

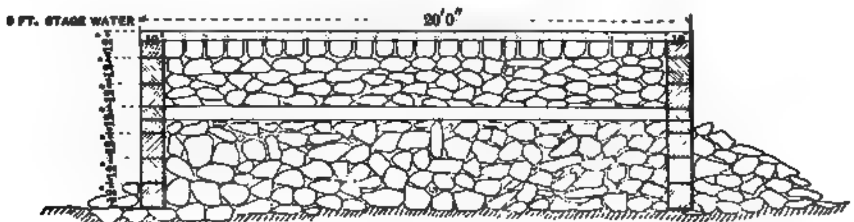
4 3 2 1 0 2 4 6



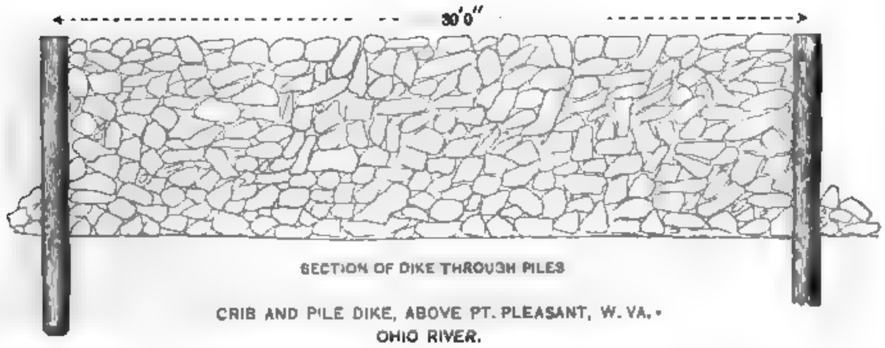
CLOSING DIKE AT WHEELING ISLAND, OHIO RIVER.



CLOSING DIKE AT BROWNS ISLAND, OHIO RIVER.

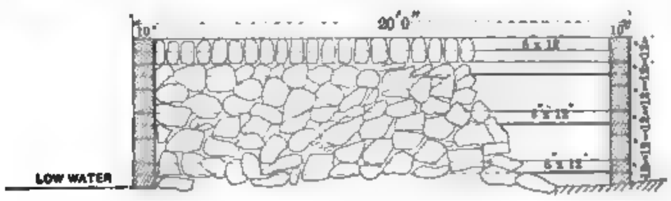


SECTION OF DIKE THROUGH CRIB



SECTION OF DIKE THROUGH PILES

CRIB AND PILE DIKE, ABOVE PT. PLEASANT, W. VA.
OHIO RIVER.



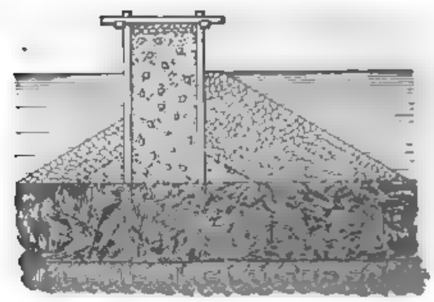
BONANZA BAR DIKE, PORTSMOUTH, OHIO.
OHIO RIVER.

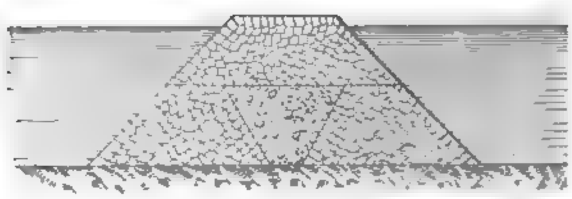
SECTIONS OF DIKES ON
FRENCH AND GERMAN RIVERS.

(From Rivières à Courant Libre and The Engineer)

SCALE OF FEET

10 0 10 20





(To face

stones which can be handled by a man without additional appliances, but it should be covered and protected with stones which cannot be easily moved, and the heavier they are the better for the stability of the dike.

In the construction of a loose-stone dike a good stage of water is desirable in order that boats may float over the site to deposit the stone, but the building of a crib dike will not infrequently be facilitated by an ordinary stage. In this structure it is customary to build the cribs in sections in the water immediately upon the site, or to tow them to the location after they have been put together, when they are sunk upon the river-bed and filled with stone or gravel, or both. As in the case of loose-stone dikes it is advisable to place heavy stones on top or to deck with plank, the latter method probably being more generally applied.

The construction of pile dikes is very simple, consisting of little more than ordinary pile-driving, and the dumping of loose stone from a scow.

Closing Dikes.—Rivers often divide into two or more passages. The effect of this bifurcation is to diminish the flow; and to place the stream in a condition more favorable to navigation at low-water stages it becomes necessary to close the additional arms. This is generally accomplished by the construction of low dams or dikes across the chutes, several being sometimes required in a single channel. The construction of these dikes presents some special difficulties, not encountered in dikes along a shore. Where built of stone the method is as follows: A layer of stones is placed upon the site of the proposed work, extending sufficiently down stream to act as an apron to receive and carry off the water overflowing the dike, and a sufficient distance up stream to form the floor of the structure. A rule given by an authority* for the width of this bed of stone is that it shall be fifteen times the fall produced. Its thickness must be determined with reference to the character of bed upon which it is placed, for when the bottom is soft it is liable to sink. Upon the bed thus prepared the main structure is built, in layers extending its entire length. The up-stream slope is about 3 of base to 2 of height, while that of the lower side is generally less, being about 2 to 1 for low dikes, and decreasing as they become higher. As the construction proceeds the water will rise over the structure and produce settlement at various points, or possibly along the whole dike. Additional material must at once be put in in order that the profile may be maintained at the desired elevation; otherwise a loss of some of the material already in position may follow through undermining. As the water rises above the dike its force increases up to a certain limit, after which the overflowing sheet loses its scouring force. It is, therefore, advisable to proceed with great rapidity as the height of the works is raised. The paving or coping should be of selected stones, hand-placed and roughly jointed, in order that the surface presented to the overflow may be uniform and solid. Piles can be used to good advantage in a foundation for this character of structure, and effect a saving in stone.

* De Mas.

Where the channel is to be closed by timber-cribs filled with stone, the general method of construction is the same as described for the building of fixed dams, and care must be taken to prevent the water from cutting under or around the structure. The elaborateness of the precautions will of course depend on the nature of the banks and bed, and the volume of water to be controlled.

CHAPTER IV.

PROTECTION OF BANKS.

Objects.—The purpose of the protection of banks is the prevention of erosion, and may include one or more of the following objects:

1. The reduction of the quantity of traveling sediment and consequent lessening of deposit in the river-bed.

2. The maintenance of a permanent minor bed of normal depth and width in the bends.

3. The protection of property, wharves, landings, etc.

4. The protection of levees built along the banks.

5. The prevention of cut-offs which may affect a river's régime and also leave important commercial centers without means of transportation.

Rivers as a rule have not a sufficient capacity of bed to carry off their waters at flood times, and this leads to a constant effort at widening by cutting away the banks, particularly on the concave sides at bends. The material thus eroded is moved along in the channel until it finds lodgment, when it obstructs low-water navigation in the form of a bar or shoal. As the bank is torn away the river shifts its position toward that side, and year by year its low-water channel, as well as its bank line, is moving. The floods undoubtedly scour out the bed at the same time they cause the banks to cave, but by reason of this continual shifting and alteration of channel this scouring does very little good. That done one year may be obliterated the next, and a new chute cut out nearer the bank. The result is that the great amount of work done by the river itself toward providing a good channel is ineffective, and it is necessary, in order to take advantage of this work and to reduce the quantity of material to be transported, to permit no encroachment upon the banks nor movement of the channel. In other words, the shores must be held to a fixed line, and thus give a permanent, if restricted, passage for the water, and one which must not be allowed to silt up with material cut from the banks.

Character and Kinds.—It is important in laying out protecting works that they should be planned so that they will not modify too greatly the régime of the river and thus bring currents upon points heretofore uninjured. They should be strictly defensive, because, while riparian owners are compelled to endure losses from floods, they have good grounds for complaint when the construction of works brings serious injury upon them. In fact the Government is often held responsible in the public mind for

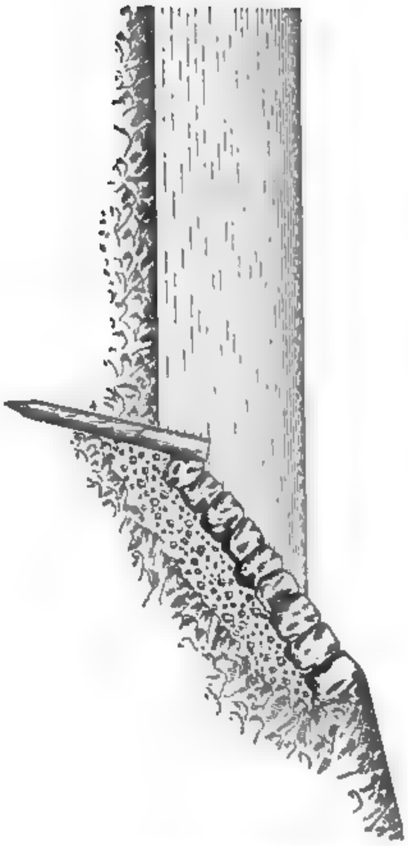
damages resulting entirely from causes over which it has no control, if such injuries have occurred in proximity to public works. It is thus of first importance to have a thorough knowledge of the reach to be protected, as time and money both may be wasted through a lack of acquaintance with the currents, etc.

The revetments constituting the protecting works of river banks comprise two distinct forms. In the one the bank is covered completely for the entire length of the space to be protected; in the other, the space is divided into sections by separate revetment-dikes placed at considerable distances apart, the bank between being left uncovered. The revetments themselves comprise two distinct parts, one below and the other above low water; the first is inaccessible and invisible, and must support the second, which is exposed to all the changes of weather and water, and is open to inspection during the dry season. As the portion under water must act as a foundation for that higher up and cannot be readily inspected and repaired, it should be built in a substantial manner. A stable foundation is a requisite for good bank protection, since if it is composed of soft or perishable material the subsidence of the upper part will assuredly result. It may be built of stones thrown into the water and allowed to take a natural slope. In many cases the first stones placed will settle into the river-bed, but this settlement will eventually cease as additional material is put in. To avoid this subsidence it is a common practice to dredge a trench into which the stones are dropped, forming a toe. This, if carried to a sufficient depth, insures at once the stability of the revetment, and usually requires less material for its accomplishment. It is needless to remark that large stones are better than small ones for this purpose, particularly along the water face.

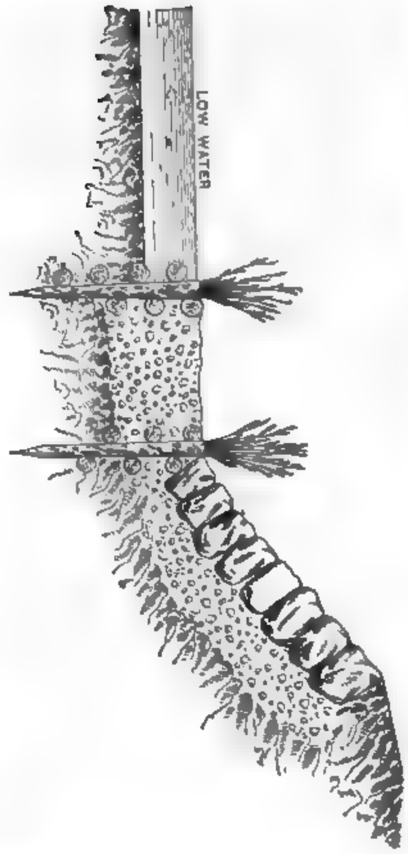
The use of round and sheet piles in connection with broken stone is quite common also in these foundations, and their employment usually lessens the quantity of stone required, and permits the work above water to proceed without danger of future settlement, if they are driven well below the limit of erosion. A base is obtained in this way which is capable of resisting pressure from both sides, and of assisting in distributing the loads over soft soils. Whatever woodwork is used in the foundation should be constantly submerged, in order to avoid the alternations of wet and dry; if timber must be used in exposed positions it should be only in those where it can be seen and repaired during each low-water season without disturbing the foundations.

A revetment of dry masonry, or of masonry laid in mortar, is sometimes used as a foundation, and as a protection above low water as well, but it is more expensive than loose stone and timber, and in the majority of cases more difficult to build and to maintain. The difference in cost between dry masonry and masonry laid in mortar is not great. Brick and concrete are also employed for the purpose of bank protection. In all these forms it is necessary to prepare a foundation-apron of broken stone or gravel which will prevent erosion.

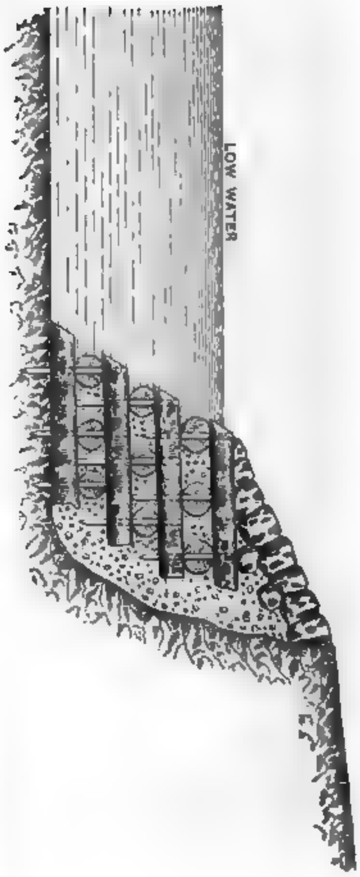
Masonry protection works, whether of stone, concrete, or brick, are not generally employed on account of their cost, except at cities or great industrial works. They



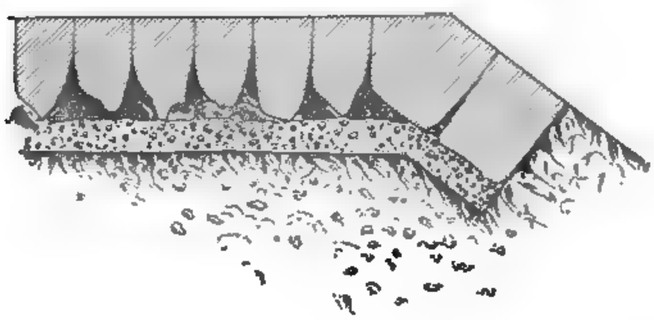
Stone Paving, with Piles.



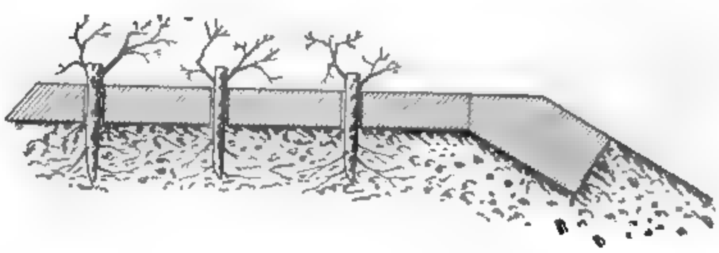
Hurdle-work and Stone Paving.



Cribwork of Fascines.



Stone Paving on Gravel Bed.



Stone Paving, with Willows.

SECTIONS OF BANK PROTECTION.

(To face p. 69.)

give satisfactory results, both in permanence and in appearance, and a less hold for the action of the water, and present no projections to damage craft. They are tight, but where on a soft foundation a settlement of earth underneath and behind them often occurs and causes a sinking, breaking their continuity, and not infrequently resulting in serious consequences. This is particularly noticeable in masonry where mortar is used. The solidification of the mass by the mortar permits a large area of earth underneath to give way before failure, and thus there may occur without warning a break of considerable dimensions, and one which may be left unprotected, or be even unknown during lengthened periods of high water. Where there is no mortar in the joints the paving will generally follow the sinking of the ground and keep it covered, and, if not submerged, will at once reveal what is taking place.

In order to reduce the quantity of stone required in a given area the pieces are sometimes placed flat, in thin layers, which are held in place by vegetation. This method has been used in France to a considerable extent. Usually flat stones are used for this work, but along the Meuse "dog's-heads" (cobble-stones) are frequently employed. The method generally consists in placing stones side by side on a graded bank, the joints being filled with thin slips of willows. If the work is done at a favorable time the willows at once take root and protect the stones by both branches and roots, the latter at the same time penetrating the soil and thus forming a perfect connection between the bank and its protection. These works are said to resist ice, the great enemy to banks in many localities, better than the masonry protection above described. In order to keep the willows in bush form and thus prevent the growth of obstructing trunks it is necessary to cut them from time to time. It is claimed for this form of revetment that it is at once an economical and efficacious mode of protection.

Excellent results are obtained by using various forms of brush, poles, etc., in protecting banks in all countries. These materials are cheap and usually grow in profusion near the points requiring works of defense. They are employed in a great variety of forms but principally in the shape of fascines and mattresses. The former are bundles of flexible branches held together by wires or other ties. They are placed close together, either singly or in horizontal layers over the bank to be protected, to which they are held by stakes driven into the soil. They may be woven into mattresses by means of poles, and when so arranged they form an excellent and impervious covering of great strength, and gradually become consolidated by deposit.

The fascines used in Holland are 8 to 13 feet in length, and from 1 foot 4 inches to 1 foot 8 inches in diameter. On the upper Rhine they are from 13 to 16 feet in length, and from 1 foot 2 inches to 1 foot 10 inches in diameter.

The mattresses are made in numerous forms and by several methods. They consist essentially of poles, brush, branches, etc., woven together with wire, or fastened with timber or ropes, sunk into position and held there by stone. In this country, of late years galvanized iron and silicon bronze wire have been used for binding, ordinary wires having soon rusted out and permitted the mattresses to go to pieces. Wooden

pins have been largely employed for fastening binding pieces, and form a much more satisfactory method than wires in swift currents.

Mattress revetment is the chief method employed along the Mississippi and Missouri rivers. There brush grows in abundance, and in spite of continued denudation for these works the supply has not been exhausted, as cotton-wood and willows spring up rapidly, so that it is the cheapest material for use. Out of the abundance and cheapness of this material has grown the practice of its use, in connection with stone, also fairly plentiful, as a revetment for banks in this country.

In regard to these works it is stated * that "The bank revetment work (on the lower Mississippi River) is probably more extensive than any like engineering construction in the world. A mattress 300 feet wide by 1200 feet long represents a superficial area of about 8 acres, and when one realizes that this vast willow carpet, over a foot thick, is placed on the bottom of the river in depths of from 40 to 100 feet, and against currents of from 5 to 8 feet per second, the difficulty of the enterprise will be appreciated. Though much of the revetment from Cairo to New Orleans has needed repairs from year to year, and in some reaches has required renewal as a whole, it may be said to have been eminently successful in the protection of harbor fronts and the prevention of cut-offs and outlets, and fairly so in the control of bank-caving, and the resulting change in position and flow of the river.

"At some points, where the material of the bank was friable and the currents very strong, the earlier forms of revetment proved too light and were entirely swept away, the shore line continuing to move back. Also considerable reaches of protection work needing repairs and reinforcement at the ends have been destroyed because of the lack of funds, due to the failure of appropriations, etc. But in the later work the results have been beneficial and satisfactory, and the loss but slight."

It is not customary to carry the protection to the full height of the bank, particularly where this is above the highest floods, and recourse is had in the upper portions to sodding. In America this has been done only to a very limited extent, but the practice is quite common abroad. This kind of revetment is made by means of pieces of sod cut into squares or rectangular figures and placed in courses normal to the slope where the latter is steep, and parallel to it where it has a gradual inclination. In certain localities Bermuda grass has been used, the sprigs being placed from 6 inches to a foot apart. As this grass possesses a phenomenal vitality, the roots spread rapidly and form a dense sod in the course of a year or two, completely covering the bank.

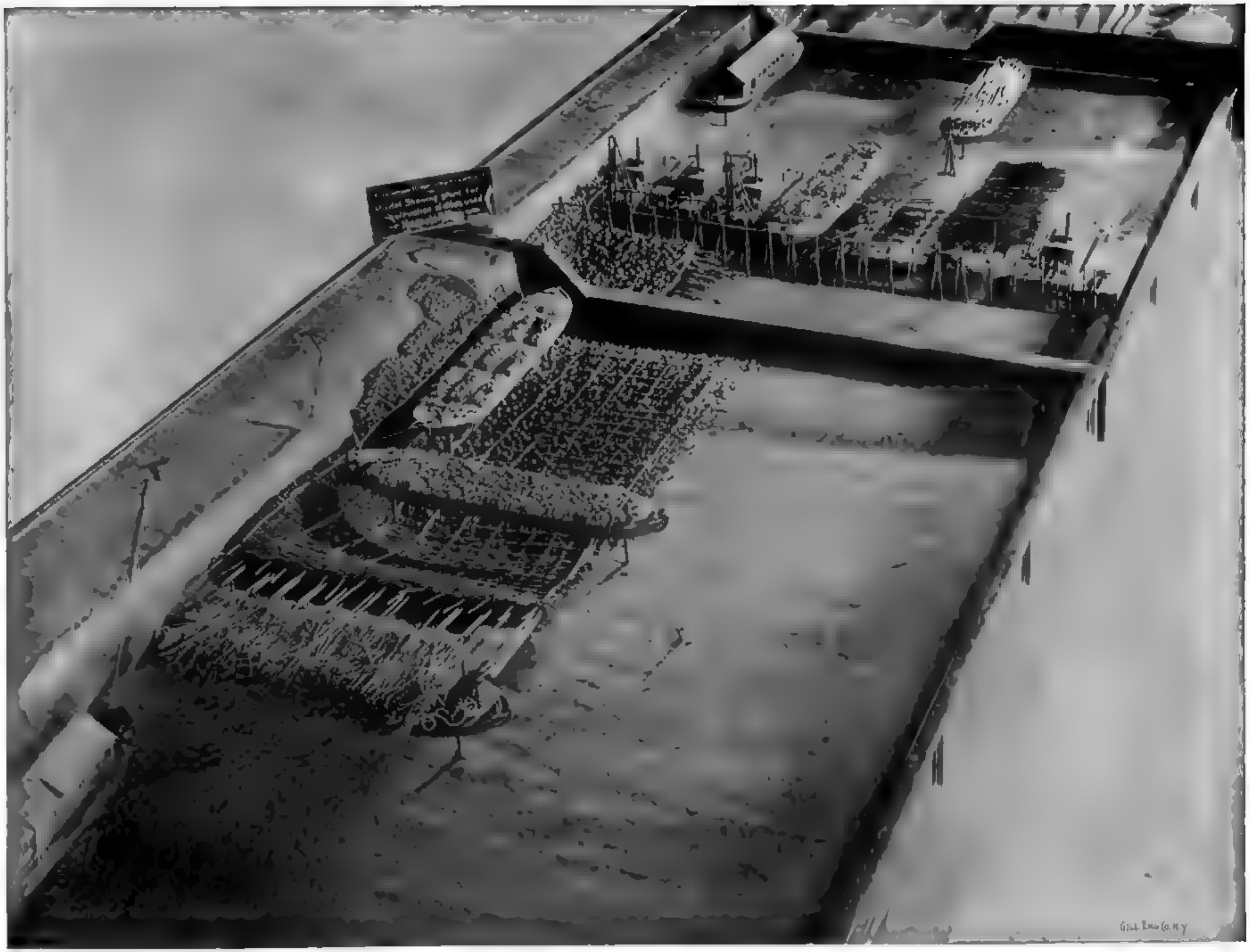
As has been mentioned, one form of protection, called continuous revetment, contemplates the entire covering of the section of bank under treatment, while another, known as spur-revetments and bank-heads, has in view the covering of isolated portions only, depending upon the works for warding off damaging currents along the spaces between them. The continuous revetment is in most general use, both at home and abroad, but there are a number of examples of the other form along the Missouri.

* Bank Revetment on the Lower Mississippi, Am. Soc. C. E., 1896, H. Coppée.

WOVEN MATTRESS UNDER CONSTRUCTION.

WOVEN MATTRESS JUST BEGINNING TO SINK.

(*To face p. 70.*)



MODELS OF SHORE PROTECTION, PILE DIKE, AND PLANT.
Improvement of the Mississippi River between the mouths of Missouri and Ohio Rivers.

(To face p. 72.)

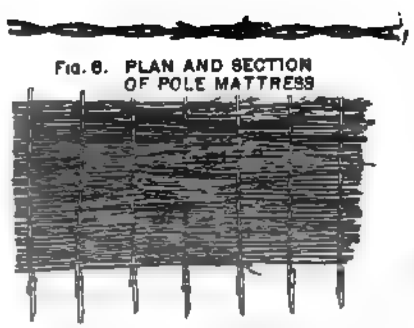


Fig. 6. PLAN AND SECTION OF POLE MATTRESS

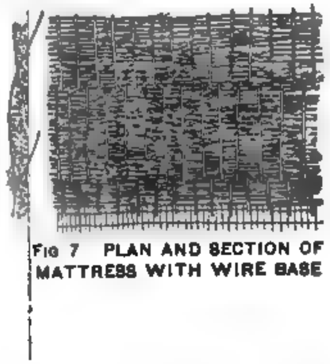


Fig 7 PLAN AND SECTION OF MATTRESS WITH WIRE BASE

POLE AND WIRE MATTRESSES.

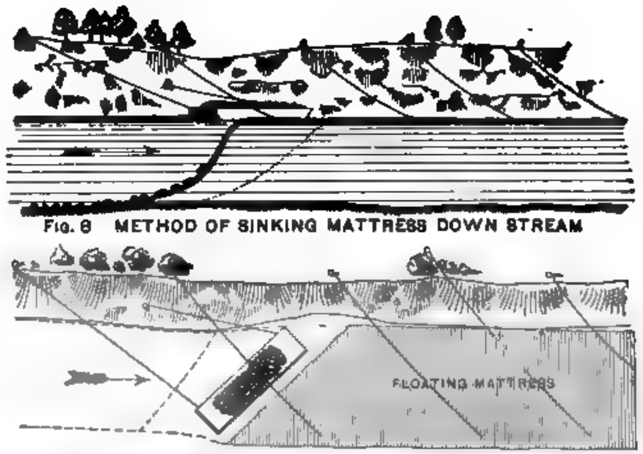


Fig. 8 METHOD OF SINKING MATTRESS DOWN STREAM

Fig. 9 METHOD OF SINKING MATTRESS UP STREAM

METHODS OF SINKING MATTRESSES.

(To face p. 71)

Construction.—The revetments of the Mississippi River below Cairo are usually constructed as follows: The bank is first cleared for a distance of 50 feet back from the top and is then graded on a slope of 4 to 1, measuring from the low-water line, by hydraulic processes. This is followed by a dressing of the new slopes which includes the filling of holes and the removal of snags, stumps, etc. A cluster of piles is then driven near the upper end of the space to be covered and at zero line, and below this cluster, or abutment as it is called, single piles are driven at intervals of 100 feet for the full length of the mattress, in order to keep it over the zero line, during a limited fluctuation of the water surface, while being built and sunk into position. The mattress is built on barges placed end to end and near the shore, up-stream of this abutment and row of piles. These barges lie outside of others, called mooring barges, which are tied to the shore by wire lines. The method of construction in detail as shown by a Government report* is as follows: Hardwood poles, as large as can be conveniently handled by a gang of men and reasonably straight, are laid in two lines on ways over and parallel to the inner gunwale. These poles lap each other 10 to 15 feet, the two lines breaking joints. Where they lap they are spiked together, and they are also tied together with No. 12 galvanized wire at intervals of 10 feet. Two ties are made at the laps. This line of poles is as long as the mattress is wide. About 7 feet 6 inches apart on these poles and at right angles to them the butt ends of weaving-poles made of live willow or cottonwood brush from 4 to 6 inches in diameter and 25 to 30 feet long are fastened with spikes and wire. Another set of poles similar and parallel to the first are placed on these and securely spiked and wired. To facilitate weaving, the tops and the bottoms of the weaving-poles are shaved and the knots trimmed. A cable made of eight strands of No. 12 wire is fastened around the head of the mat at every third weaving-pole and run up alongside of it, the end being fastened thereto by two staples. These cables are 24 feet long, with an eye in one end, to which, after each shift of the mat, a new length is looped in weaving. Ten continuous cables are thus formed in the mat, greatly strengthening it longitudinally. When this head is finished, lines are connected to it from the shore, passing under the mooring barges.

The brush used for weaving is live straight willow of any length over 25 feet and from 2 to 4 inches thick at the butt.

The butts are placed over one weaving-pole and project 2 feet beyond, being woven at the other end over the next pole, under the third, over the fourth, and so on, the light ends being always left on top. A strip 5 feet wide is thus woven. In the next strip the butts are reversed, the butts changing directions every 5 feet. When the mattress is woven within 2 feet of the end of the poles, giving about 22 feet length of mattress, it is swung in position with the accompanying barges. The head-lines on the barges and mattress are slackened until the barges are nearly normal to the shore, with their inside edge resting against the pile abutment. The slack in the mooring

* Report of the Chief of Engineers, U. S. Army, 1891, p. 3606.

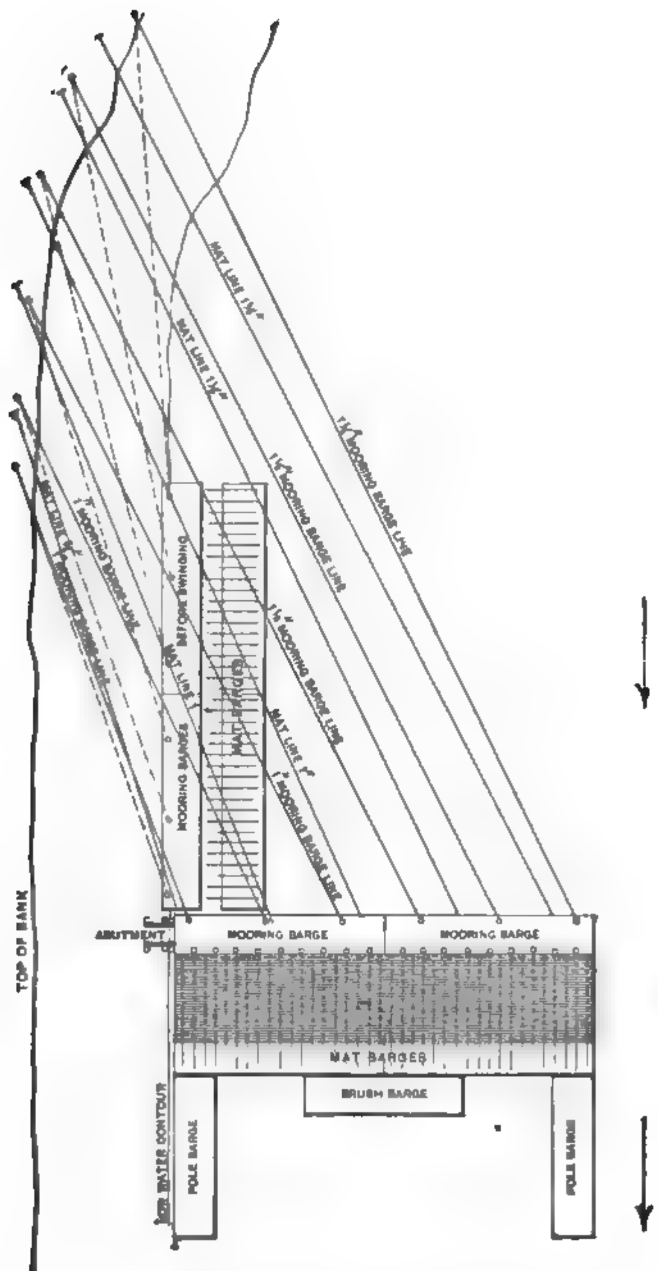
barge cables is now taken in from the bank and the strain equalized. They are then fastened permanently with clamps, as are also the mattress head-lines. An entire shift 22 feet long is then launched, a new set of weaving-poles being spliced to the projecting ends of the first set. This is continued as described to within 2 feet of the top of the second set of poles, when another launch is made, and so on, until the full length of the mattress is obtained.

When three shifts have been launched the construction of a top grillage or framework is begun. This consists of a line of poles laid over and parallel with the weaving-poles, lapping each other, butts to tops, from 6 to 8 feet, and wired to the weaving-poles every 4 feet by lashings 2 feet long, made of two strands of No. 10 wire; transverse poles 8 feet apart for the first 100 feet, and thereafter 16 feet apart, are placed in similar manner and fastened to the longitudinal ones at the intersections by two-feet lashings made of four strands of No. 12 wire.

The purpose of the grillage is to make pens in which the stone is retained, as well as to strengthen the construction. The first set of transverse poles along the inner edge are hardwood, set 8 feet apart throughout the length of the mat, and are used to connect the shore mat, which is also being built, to the river mat.

The construction of the shore mat is as follows: Hardwood poles of the size of the weaving poles are lashed and spiked to the river mattress, and willow or cotton-wood poles are spliced to these until they reach up the slope about 40 feet. Alongside and fastened to each of the hardwood poles is a cable made of eight strands of No. 10 wire, one end of which is fastened to two of the adjacent weaving-poles, and the other to the willow poles extended on the slope. Upon the transverse poles are laid longitudinally willow or cotton-wood poles 8 feet apart, beginning with the first set about 4 feet from the edge of the mat. The latter poles are wired to the former at their intersections. The longitudinal poles are carried on lines 8 feet apart up to the top of the slope, and on their lower side, 8 feet apart, are driven stakes 2 feet 6 inches above the ground, to the tops of which is loosely fastened a lashing of wire whose bight has first been passed under the pole. These stakes are used down to the pole nearest the water edge. Upon this framework is laid willow brush diagonally with the butts toward the top of the slope and breaking joints throughout. A second layer of brush is put on in the opposite direction, the two thus being at right angles to each other. On top of these layers a second pole framework, fastened similarly to the first, is placed and fastened down firmly by the lashings mentioned above as being tied to the stakes. As fast as the river- and shore-work is finished transverse cables are run across the entire width of the mat at 16-foot intervals, carried to top of bank, and hauled taut and fastened to trees, stumps, or deadmen placed for the purpose. These are fastened to the mat every 16 feet with lashings.

When 400 or 500 feet of river mattress have been completed, longitudinal cables are run out from the mooring barges and securely attached to the mat at 16-foot inter-



Position of Barges During Mattress Construction.

(To face p. 72.)

Inclined Ways for Fascine Mattress Construction

Fascine Mattress, Completed. 300×1125 Feet.

(To face p. 73)

vals. One of these is placed close to the outer edge of mat, the others at 30, 37, 38, and 42 feet respectively.

Ballasting can begin after 600 feet have been finished. If the mattresses are not to be more than a thousand feet in length no ballasting need be done until it has all been completed. The stone is wheeled from barges and placed along the transverse poles, loading the entire floating mat until only the poles are above water. By then bringing the stone barges immediately over the mattress and unloading them the structure is sunk to the river-bed. The shore mattress may be ballasted at leisure from barges or otherwise.

Owing to scour along the outer edge many of the mattresses of this type were damaged, and other defects also developed. In order to overcome these difficulties a more flexible and durable form of mattress has been used for the outer edge, composed of fascines or bundles of brush 1 foot in diameter and in lengths of 50 to 100 feet, tightly pressed and bound together at 3-foot intervals.

The regular fascine mattress may be constructed in two different ways, one with the fascines normal to the bank and the other with them parallel to it. The latter is generally considered to be the more flexible type. The fascines are made by putting the brush in two layers with the butts in opposite directions, always breaking joints. These layers are drawn together by chains at 8-foot intervals, and then bound by wire into a bundle about 12 inches in diameter. These are then woven into mattresses and sunk in a manner not unlike that described for the ordinary woven type. The slopes above the mattresses are usually paved with stone.

Surveys of revetted reaches seem to prove that where the bank is protected, no matter how strong the revetment, the channel is deepened just outside the subaqueous work, and under its outside edge, causing it to take a steep grade. If the mattress is built with sufficient flexibility, strength, and compactness, its edge will slowly settle, and the ultimate result will be the steepening of the subaqueous slopes, without destroying the efficiency of the work.

Spur Revetments.—In 1884 a continuous revetment in New Orleans harbor was broken up by the river after sinking, and this led to the introduction of submerged spurs normal to the bank and placed at intervals of from 500 to 1600 feet, usually at salient points. These structures consist of a woven mattress foundation of the width deemed advisable, and extending out into the stream usually beyond the deepest water and protected on the edges with a narrow cribwork of willow poles filled with rock. In the case mentioned additional cribs were sunk one on top of another, at a distance of about 70 feet from its up-stream edge, and affording a base of about 60 feet and a top width of about 22 feet. This cribwork was about 300 feet long. Similar work was put in at Memphis later on and at other places.

After sinking the mattress and cribs the portion of the bank opposite to and above it should be graded to a flat slope and covered with a revetment of willow and stone,

connected with the submerged mattress. Frequently a bank crib is put in also, connected with the submerged cribs.

The efficiency of this type of revetment when placed in caving bends is to a great extent dependent upon the radius of the bends, the distance between them, and the material of which the bank is composed. They are stated to be of little value in very abrupt bends with sandy banks, and that where the banks are of clay and "buckshot," the distance apart should not exceed 500 feet, and even then it may be necessary to protect the intervals between them.

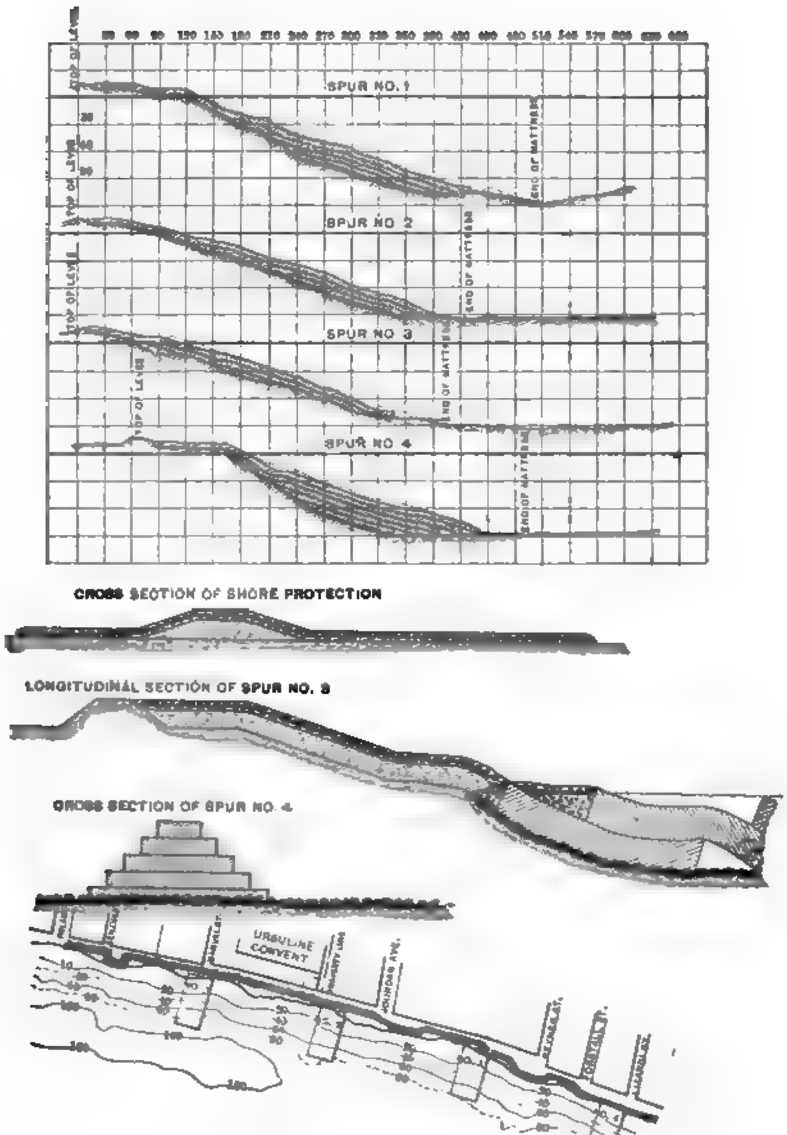
Bank Heads.—On the Mississippi and Missouri rivers there has been designed and constructed in recent years, by Col. Amos Stickney, Corps of Engineers, U. S. A., a form of revetment somewhat similar to the spur-revetment, and known as a "bank-head," which consists of an isolated section of bank protection, of a size and distance apart depending on the local conditions. It is placed on concave bends, and at the edge of the bank. Its use is based on the theory* that by holding permanently certain points of the bank at certain distances apart the force of the current will not be able to seriously cut in between these protections, nor cut around them, owing to the shortness of the space in which its effects, such as scour, eddies, etc., have to work. On the concave side of a sharp bend, for instance, if the points are too far apart, the velocity of the current will sweep against the bank between, and cut it away until it can attack and undermine the bank-head, but if the points are at the proper distance, the current will work into the bank for a certain distance, and the erosion will then cease, as there will not be room enough for further action.

Experiments on the Missouri River showed that a considerable current of water will pass around a fixed curve of 300 feet radius without causing violent eddies, and that an angle of 30° to the current is approximately the one at which a soft bank can approach a fixed point without much erosion. With these data as a basis the bank-heads are built with a conical front, and a least radius of 300 feet. They are constructed of ordinary materials, as brush, stone, etc., and provided with an ample protection of riprap on the upper and lower sides. A large mass of riprap is also placed along the river-face, and renewed as the water undermines it, and it has been found that this action ceases after a certain time, as the stone having fallen over will gradually afford a protection to the bank below the limit of scour.

As far as they have been used these bank-heads have afforded very satisfactory results.

---

* Annual Report Chief of Engineers, U. S. A., 1897, p. 3537.



New Orleans Spur Revetments.
(For Bank Protection.)

(To face p. 74.)

CHAPTER V.

LEVEES.

History.—Levees are embankments of earth thrown up to prevent overflow from streams, or to stop the sea from inundating adjacent lands. The Egyptians and Babylonians were the first of whom history speaks as having embanked their lands, and they were followed by the Phœnicians, the Romans, and East Indian nations. One of the earliest examples of levee building was around the city of Babylon. The Euphrates was embanked on each side, the embankments leading to the bridge across the stream. The skill displayed by the ancient Romans and others, and the extent to which they went in embanking and reclaiming marsh lands are shown in the levees along the Tiber near Rome, the Po near its mouth, and in the Fen-lands in England and in Holland and other countries, but the attention of scientific men was not brought seriously to the problem until Italy began systems of levees along its rivers during the thirteenth century. The Arno, Tiber, and Po were partially embanked, followed in the seventeenth century and later by the Chiana, Adige, Reno, and many of their tributaries. The discussion brought about by the prosecution of this work resulted in a general levee system, and enlisted some of the greatest philosophers of the time among whom were Galileo, Poleni, Torricelli, and Zendrini, and the result is that to-day these rivers are confined between embankments which, although artificial, are centuries old, and which have served as examples to Holland, Spain, France, Germany, Ireland, England, and the United States, all of whom have profited by the systematic works along the Po. The vast quantities of alluvial matter brought down by the Rhine, the Maas, and the Scheldt formed salt marshes which were later reclaimed by means of levees, known far and wide as the "Holland Dikes," and out of this marine swamp arose the rich kingdom of Holland. There are levees also along the Rhine, the Oder, the Elbe, the Vaert, and other rivers of Germany and Holland; along the Thames, Mersey, and others in England, the Loire in France, and the Vistula and Elbe in Prussia.

In the United States, while there are numerous levees in various parts of the country, and some of them of importance locally, there is but one extensive example of levee work which can claim attention, that of the Mississippi and its tributaries below Cairo. Prior to 1860 this important work was carried on in the States of Arkansas and Louisiana by the State governments, while in Missouri, Tennessee, and Mississippi each county bordering on the river had charge of its own levees. The results were very unsatisfactory under county governments, and, even where directed

by the State, conflicting interests, lack of knowledge, and a variety of causes conspired to render the work expensive and not always of the greatest benefit. Then came the Civil War with its devastation and derangement of conditions, and the levees were virtually abandoned or wholly destroyed. Attempts were made by the local governments later on to repair them and even to build new ones, with varying success, until finally, about 1880, the Federal Government took the matter in hand. The State governments of Missouri, Arkansas, Mississippi, and Louisiana, however, continued their works in certain localities in coöperation with the United States, since which time marked improvements have been made in the design and methods of construction.

Reports show that more than $15,000,000 have been expended on levees by the United States.

Location.—Permanence, economy of construction and of maintenance, and future enlargement are involved in the location of a system of levees, but it is a rare thing to see a location made with these objects solely in view. Too often the interests of the local property holders are the first consideration. While these should be recognized to a certain extent, the general benefits to be derived from properly located lines should always be considered first. As the systems are extended and completed the flood-plane will rise, necessitating new work which, if carried on with locations improperly made, will mean the expenditure of vast sums of money. It is far better to expend that money in first constructions so located as to give the best protection to the greatest number, even if such protection damages the few. In the original alignment of the levee systems along the Mississippi no attention was given to the laws of motion of fluids; levees wound around every cow-pen and horse-lot, presenting obtuse angles at critical places without additional thickness of section. These locations have still been adhered to in many cases in the enlargements made by the General Government; in many other cases the embankments have long since gone into the river with caving-banks or have been weakened at their salient angles and destroyed by the floods.

Angles should always be avoided and curves substituted flat enough to admit of a railroad track being operated. Hewson* lays down the following rules for locating a levee: "The first duty is the mapping out carefully of the bank, and, as far as may be done by a careful sketching, of the current set, the 'caving,' and the 'making.' In the case of cavings and makings, every information as to their commencement, their rate of progress inwards, and their advance down stream should be obtained carefully from local information and recorded at the proper points on the map. The cavings and the makings of the bank pass down stream in a series of waves, period after period; and therefore by ascertaining the rate of descent, the rate of penetration of a 'cave,' or the extension of a 'make,' at the point of its operation, the location of the levee opposite that point may be made with a full knowledge of the conditions of its permanence." The following notes from the "Manual" of the Dutch engineer Storm-Buysing may also be quoted: "The trace of the new dike should of course have regard

* Embanking Lands from River Floods.

to economy of material and capacity for protection. Its general direction should be nearly parallel to that of the stream, but should avoid as much as possible exposure to the most destructive and prevailing winds. The fitness of the foundation should especially be regarded, and good high ground selected, free from creeks or sloughs. Sharp angles are to be condemned, especially re-entering angles, because they make basins in which the wind has full play to raise the water. When changes of direction occur they should be made by a curve, uniting the two tangents."

Section.—The cross-section of a levee will vary with its location, the character of the foundation upon which it is to be built, and the material of which it is to be constructed. In exposed positions, where the waves are liable to make inroads, the section of the levee must be greater and the slopes less steep than in those places where wave-action is not expected, down to solid earth. Lastly, the material of which an embankment is made will in a measure control its profile. If it is to be of sand or light porous soil it will be much more liable to be washed and cut away than if built of clay or gravel, and hence must have a greater thickness.

In this country the most usual dimensions are 8 to 10 feet across the top or crown, with slopes of 3 to 1, an increase being made for levees built of sand, in which the width sometimes reaches 15 feet on top with slopes of 5 to 1. When the levees are high a terrace or "banquette" is usually built on the land side for the purpose of obtaining the necessary strength. This bench is about 20 feet in width, sloping off more gradually than the main embankment, and is usually about 8 feet below top of the levee.

The levees along the Po are generally from 23 to 26 feet on top with slopes from 2 to 1, to 3 to 1, and usually have two horizontal terraces on the land side. The dimensions of course vary considerably with the conditions, in some places the top width being reduced to 16 feet. When used for roadways, a custom which is quite general there, the crown is of gravel.

On the Rhine the levees have a top width of about 6 or 7 feet, with slopes of 3 to 1. These narrow crowns are doubled when it is desired to use them for wagon roads.

In Prussia the levees along the Elbe have dimensions about the same as those on the Rhine, with banquettes on the land side where the height renders additional strength necessary.

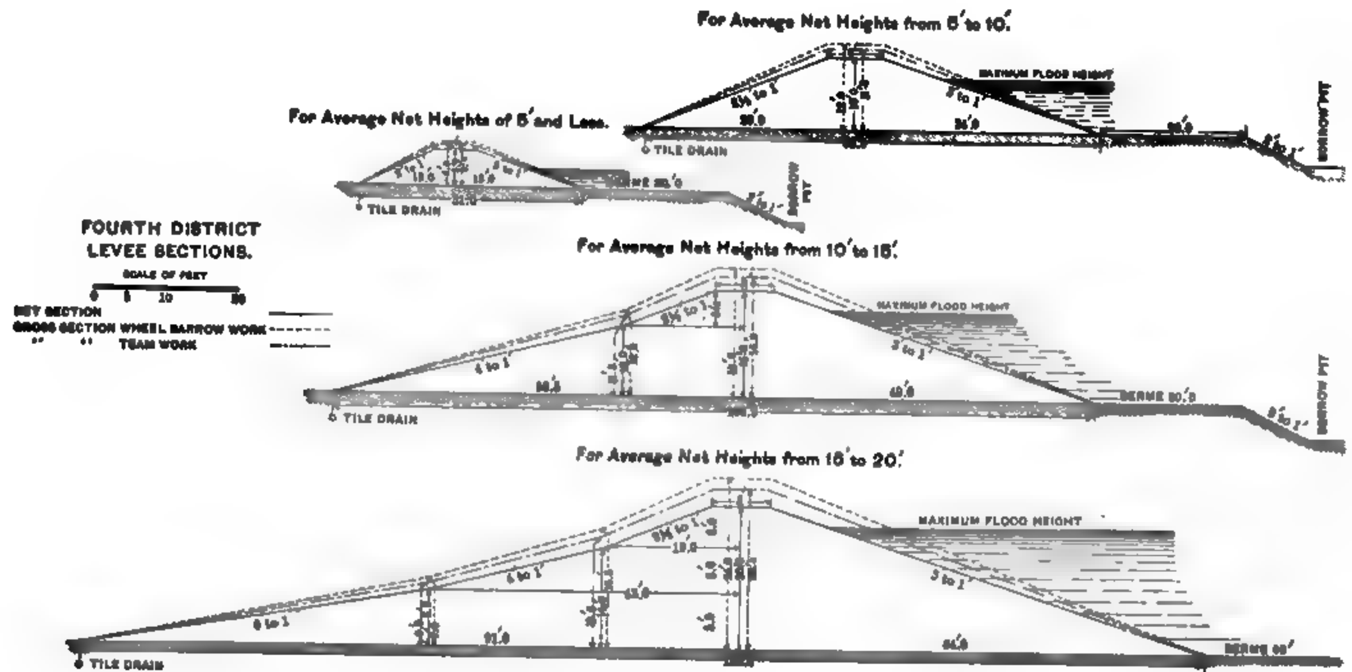
The tops of the Theiss levees rise about 5 feet above the highest flood-level, and are about 20 feet wide, with slopes of 2 to 1 on the river side above the level of the highest water, and 4 to 1 below that level, while on the land side the slope is 2 to 1. This slope is broken by a terrace 13 feet wide, placed about 3 feet below high-water level. The Vistula levees in Prussia are from 12 to 16 feet in width at the crown, with slopes of 3 to 1 on the river side and 2 to 1 on the land side. There is usually a banquette having about the same width as the top of the levee.

In Holland the sections vary greatly, according to location, material, and existing conditions. They are from 16 to 25 feet on top, with slopes of 2½ to 3½ to 1. Those built to withstand sea-waves are of course considerably more substantial than those

along the rivers, some of those in the most exposed positions having slopes of 10 to 1.

In regard to section, Hewson in his work on levees before referred to, has the following: "The presentation of equal strength at all parts of the levee does not require a greater width even under the most unfavorable circumstances than (whatever may be the proper width of crown) side slopes from each side of crown at a rate of 1 foot horizontal to 1 foot vertical. Such a section may be said to be, in general, the section of uniform strength. The strength of anything being the strength of its weakest part, an excess of strength at any one part is, it is almost needless to observe, a waste of material in leveeing, and consequently a waste of money.

"In practice, however, it is impossible to conform to the section of uniform strength in levees, seeing that the controlling consideration rests in the standing angle of the material. The standing angle of clay has been set down at 8 inches base to 1 foot in height; and, therefore, may be held to conform closely to the section of perfect economy of material—the section of uniform strength. Twenty-one inches of base for every 12 inches of height being the standing slope of sand, that material is seen in the excess of its natural section over the section of equality of strength to involve in leveeing a very large waste of material, and, therefore, of money. In a levee having a 3-foot crown, a 21-foot base, and a height of 5.2 feet, the area of cross-section is 62.4 square feet. This levee, it must be recollected, is one of equal strength; and, therefore, measuring its effective strength by its weakest part—its 3-foot crown—we find the limit of its actual resistance to be, when made of sand, as 3 feet $\times$ 95 pounds, or 285. A clay levee of 2.11 feet crown, sloped down at the standing angle of clay to a base of 9 feet for 5.2 feet in height, contains within it the slope of uniform strength, and consequently its crown being its weakest part, the limit of its effective resistance is as 2.11×135, or 284.9. This clay levee of 2-feet crown and 9-feet base presents, then, precisely the same resistance to water-pressure as does the sand levee of the same height, having a crown of 3 feet and a base of 21 feet. The cross-section of the clay bank in this case is 29 square feet; while, as has been said above, that of the sand is 62 square feet. But practice goes still further in increasing this disproportion between the different quantities necessary in levees of sand and in corresponding levees of clay. The standing angle, as presented in theory, must be deviated from in both sand and clay in order to meet, in practice, the contingencies of floods and rains. Lighter, looser, and less adhesive than clay, the flattening of slopes in sand below that of the angle or slope of repose must be much more considerable in practice than that in the heavy concreted and adhesive bank of clay, in order to resist without endangering the effective strength or stability of the bank the active washes of rains and waves. . . . Confining the equation of the materials to the simple fact of the difference between their strict standing angles, 26 yards of clay are seen to be equal, in a levee of 5 feet height, to 58 yards of sand, in accomplishing the object of all levees, namely, effective resistance to floods."



SECTIONS OF MISSISSIPPI RIVER LEVEES FOR VARIOUS HEIGHTS.

(To face p. 78.)

In regard to the slopes given to levees in America, another author* states that "the slopes of 3 to 1 now generally adopted are the outcome of a long experience. In the early stages of levee building, much smaller dimensions were often used, but disaster was frequently the result. A steep back slope very often leads to extensive sloughing or slipping, especially if the material be at all weak. A steep front slope exposes the embankment to serious abrasion by waves. A slope of 2 to 1 is less than the angle of repose of wet earth of almost any kind. In very small levees, where the width of the crown cuts an important figure, slopes may often be reduced. Thus a levee 3 feet high, with a crown of 8 feet and slopes of 2 to 1, has a stronger section than a levee 12 feet high with slopes of 3 to 1.

"Where the exposure to winds is very great the front slope is often made as flat as 5 to 1, the back slope being then reduced to 2 or 2.5 to 1. It is found that a flat slope is a great protection against the wash of waves, and that a well-sodded 'buck-shot' levee, with a slope of 5 to 1, will stand a pretty stiff wind. If the sod be once cut through, however, and a hole made in the clay, the latter is liable to be undermined, and the superincumbent masses of earth fall in huge blocks."

The following gives information as to the sections generally used:†

"A few years ago the Government adopted standard sections which have since been adhered to, except in cases where the conditions demanded more specific treatment. The first, second, and third districts of the Mississippi River have practically the same standard for all levees on ordinarily good foundations, and when constructed of material not below the average in strength.

"The standard dimensions are: Crown, 8 feet; front or river slope, 3 to 1; back slope, 3 to 1. Where the levee is over 11 feet in height, a banquette, at an elevation of 8 feet below the top of the main levee, is added. The slope of the crown of this banquette is 10 to 1, width of crown 20 feet, and back slope 4 to 1. Where the foundation is bad, or the material weak, the banquette section, and perhaps the front slope of the main levee, is increased.

"The specifications require the levee to be constructed in 2-foot layers, with scrapers, on a well-grubbed and thoroughly plowed foundation containing a small exploration muck-ditch filled back with strong material, the best to be found in the vicinity, and sodded at 2-foot intervals with Bermuda grass.

"In the fourth district the dimensions of the standard adopted vary with the height, and are intended to conform more nearly to the supposed theoretically perfect section. These variations may be further modified, as in the other districts, when required by abnormal condition of foundation, material of construction, wave wash, etc.

"For levees from 5 to 10 feet in height, the crown is 8 feet, the river slope is 3 to 1, and the land slope 2½ to 1.

"For levees from 10 to 15 feet in height the crown is 8 feet, the river slope is 3 to

* Levees of the Mississippi, Wm. Starling.

† Standard Levee Sections, H. Coppée.

1, and the land slope 4 to 1 to within 5 feet of the crown; thence to the crown it is 2½ to 1.

"For levees from 15 to 20 feet in height the crown is 8 feet, the river slope is 3 to 1, the first 8 feet of the land slope from the ground is 6 to 1, the next 6 feet 4 to 1, and thence to the crown 2½ to 1.

"In the upper districts 10 per cent of the height, both in wheelbarrow and team work, is required for shrinkage.

"These standard sections are expected to withstand the water to within 3 feet of the crown of the levee, without excessive saturation or change of form, and to give unqualified protection under all normal conditions of foundation and materials of construction.

"When subjected to water above the 3-foot line, though they are intended to remain intact, they cannot be considered, either theoretically or practically, standards of excellence.

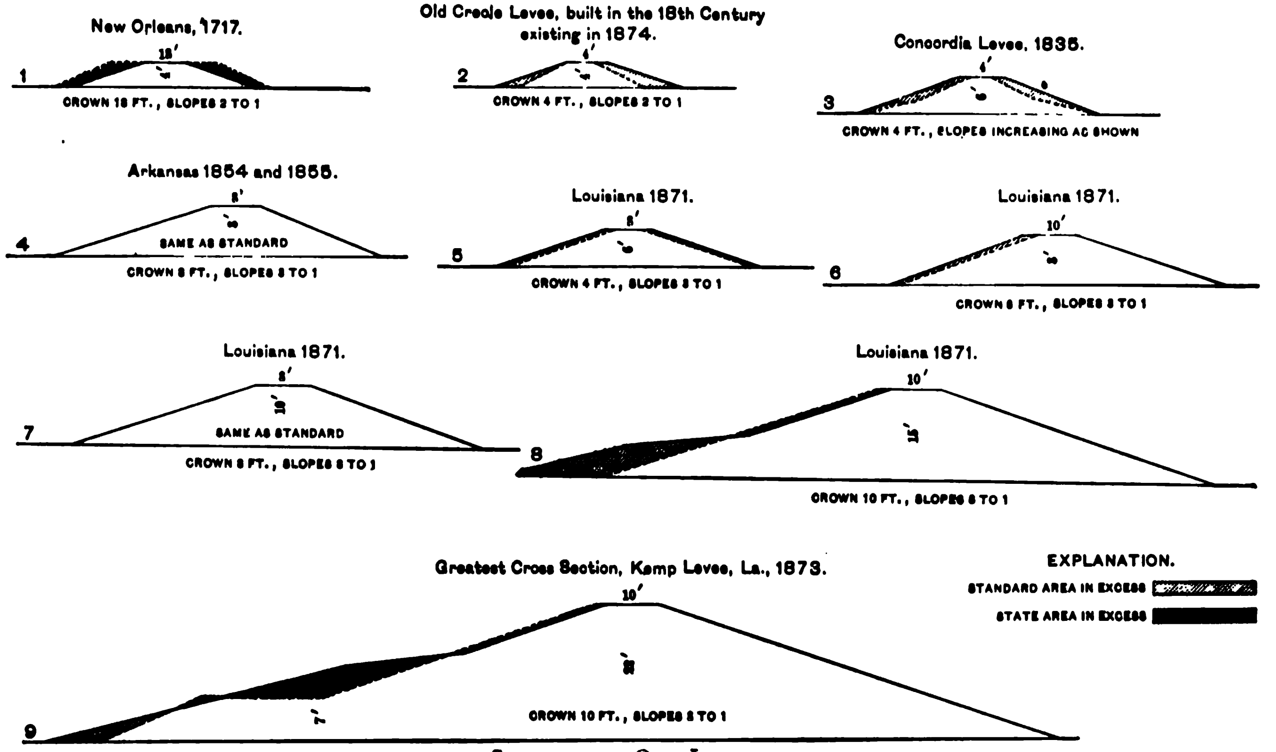
"Without taking into account the effect of waves on exposed levees, which necessitates recourse to special slopes and methods of protection, planking, revetments, etc., the whole question of standard section depends on the permeability of the embankment and foundation, that is, the extent of seepage, or percolation, and the best form and method for overcoming it in different materials.

"In 'buckshot' or clay, which is practically impermeable, the section might be given a strictly theoretical form, dependent alone on the height of the water and the weight of the buckshot; allowing some crown merely for increasing the height in time of excessive flood, the slopes being plane surfaces with an inclination sufficient to insure the required weight to counteract the hydrostatic pressure and the angle of repose of the material.

"In cases of permeable materials, light clays, sand, and loam, the levee becomes partly saturated when subjected to high water, the line of demarcation between saturated and dry soil descending in a hydraulic gradient varying in inclination with the soil of which the levee is composed, and being probably very irregular in trace because of the lack of homogeneity of the material in the body of the levee.

"In surface soils, subject to direct rainfall or percolation, from adjacent watered areas, the ground-water stands at a level dependent on the composition of the soil, both physical and chemical, the natural and artificial voids, and the hydrostatic pressure. In nearly all soils, remote from intersecting fissures, wells, or streams, the line or plane of saturation is parallel with the surface of the ground, following the inclination of hill and valley. Where wells, fissures, or river-beds occur in the surface soil, the line of moist material, or plane of upper surface of saturation, is inclined towards the fissure, well, or river, the degree of inclination depending on the consistency of the soil.

"The power of soils to resist the pressure of water is due to their specific gravity, fineness of comminution, cohesiveness, and the irregularity of individual particles.



SECTIONS OF OLD LEVEES.

(To face p. 80.)

Coarse sharp sand has greater resisting power than that composed of fine, smooth, rounded particles.

"The author estimates the strength of materials, as found in this levee district, to resist deformation due to seepage, or their value for levee purposes, in about the following order:

"(1) Buckshot and gravel tamped in shallow layers.

"(2) Buckshot artificially mixed with sharp sand in shallow layers.

"(3) Buckshot or clay.

"(4) Heavy strong soils.

"(5) Coarse sharp sand.

"(6) Light soils.

"(7) Fine sand, rounded particles."

Height.—One of the principal causes of breaks in levees is their insufficient height. No embankment of earth, and particularly of the class of earth of which levees are generally constructed, can long withstand the action of water flowing over its crest. It is, therefore, of the utmost importance to build to a height which is not liable to be reached by the greatest floods. This elevation is difficult to determine in advance, because of the uncertainties attending the coming of floods and the additional height to which the river may rise, because of the contraction caused by the levees themselves. A lack of funds has restrained the engineers on the Mississippi from building to a safe height for great floods, and they have been compelled to adopt what is called provisional grades, that is, grades adjusted to the resources. Those adopted in recent years have been as follows: Third district, 3 feet above highest flood; fourth district, 2 feet 6 inches above same; Upper Yazoo district, 4 feet above flood of 1890; Lower Yazoo district, 4 feet above flood of 1891; St. Francis district, 3 feet above flood of 1882. The flood of 1897 established a much better standard for height than had been previously available.

The effects of settlement and sloughing at various points have rendered this grade line uncertain, and in flood times it has been found necessary to add to the heights in many places in order to save the levees from destruction by overflow. The establishment of a grade which would not be reached by a river wholly confined by levees has been discussed, and in some cases adopted; but, even were the river thus fully leveed, its high-water elevation would still be irregular, as it is to-day, owing to its tortuous course and sudden changes of direction. It is evident that by bringing a levee across the course of the current the tendency to check its velocity, and thereby back up the water, will be increased, while immediately below there will be a decided fall or inclination in the surface. If a levee were to be built with a uniform grade, the water in some of these obstructed places might rise to a greater elevation than the crest and destroy or injure the levee. Mr. Starling gives * an instance of this kind where a levee several miles in length was built with an average fall in grade of about 0.4 foot to the mile,

* Levees of the Mississippi.

when it should have been built in that situation to level grade. The consequence was that the water ran over at the lower end, while it was still 1.5 feet below the crown at the upper end.

In order to successfully establish proper grades it is evident that considerable skill and experience are required, as well as a thorough study of the immediate locality. Safety will dictate a too great elevation, while economy will call for one which is scarcely high enough. There is a mean which should be intelligently thought out and applied. A Government report has the following on this subject: "The grade of levees for the improvement of navigation should be at least the normal height of the bank of the river, or the height to which the bank would be built by such floods as recur with sufficient frequency to exert an appreciable influence in bank-building or enlargement of waterway. This grade is approximately indicated by points on or near the margin of the river, well above general overflow, where deposit has been carried to a natural limit unchecked by swift currents and undisturbed by caving. This grade should be supplemented by such additional height as will protect it against frequent injury or destruction.

"Preparations must also be made in the grade for such increase of flood elevation as will at first and during the period of readjustment ensue from the introduction of greater volume between levees, when made more continuous. It is not possible to predict at present to what this will amount, and it is probable that an exact conclusion will only be reached by experience.

"We therefore conclude that levees, such as have been herein described, are, in connection with an equalization of width and the prevention of caving, an important part of any general and systematic plan for the improvement of the navigation and the prevention of destructive floods; and we recommend the construction of new and raising of existing levees along all parts of the river where the highlands are too remote to check the passage of large volumes of flood water outside the bed of the river; or, in other words, on the entire right and also on the left bank below Baton Rouge, and from the Yazoo River to Horn Lake, below Memphis."

Materials.—Levees are generally built of the material nearest at hand, whether it be loam, sand, clay, gravel, or a mixture of two or more of these. The clays along the Mississippi are known generally under the name of "Buckshot," the name being given because of its peculiarity of breaking into fragments about the size of bullets. Buckshot is not wholly clay, there being a varying percentage of sand in its make-up. The sand used in levee construction is mixed with earth and is of fine particles. The clay is usually plastic and of the blue variety, although other kinds are found. The loams are composed largely of sand, and are light and fine, and very poorly suited to embankment building. By reason of its resistance to the action of the waves, as well as to percolation, clay is the best material found for this work.

Sand makes a fairly good embankment when not subjected to wave action, but it is not safe to build one of small thickness, because a cavity once formed will increase rapidly and soon endanger the whole structure. In fact, sandy material of any kind

is unreliable and difficult to manage, and even when covered with sod is not to be depended on. Where it is practicable to place a layer of clay upon the outside fairly good work can be obtained, but when a better material can be had without too great expense the use of sand is inadvisable. Not only is the wave action severe on sand embankments, but they are liable to sink and slough as the fine particles are washed out, and once sand starts to escape it is a difficult matter to arrest its movement. Whole embankments will thus sink away without warning and with great rapidity. Clay will resist water and erosion to a much greater extent than sand, but it does not stand at grade very well. Its tendency is to settle and crack, and in this it is inferior to good sand. To make a reliable embankment it must be placed in thin layers and be well tamped as put in. It is very difficult to handle in wet weather and gets very hard during a dry season.

The fineness and lightness of loam renders it undesirable for levee work. Its particles lack coherence, and it is even more treacherous than sand, because saturation transforms it almost into mud. Where loam is used the embankments should be given much larger dimensions than with those of clay. On the other hand, it is usually very abundant, being a surface soil, and it is easily worked in dry weather, packs well, and holds up to grade.

A combination of clay and sand is highly recommended by most engineers who have had experience in levee construction. It gives a bank of a permanent nature and is also easily built. It prevents the cracking noticeable in clay embankments and does not shrink so much, and at the same time the tough quality of the clay is preserved.

One of the most important materials used for river embankment is gravel, which in many localities is easily procured. Its use is generally as a facing to prevent wash, the inner portions of the levee being of a more cohesive material.

Upon this subject Hewson says: "The lightness of a sand-bank is but a small disqualification for leveeing compared with its liability to wash and leak. Its wash is not confined to waves, current, and rain; but is carried on actively by the wind. Sand is liable not only to run and blow away in a dry state, but also in a wet state is liable to run or 'melt' like so much sugar. But while its lightness lays it open as a material for levees to great objection on the ground of duration, the worst of its properties in such works is its liability to percolation. A bank which may be of ample section to resist the total pressure brought to bear on it, when that pressure acts from the outside slope against the whole weight of the bank, will yield when that pressure becomes transferred from the outside of the bank to some point or plane within it. In the latter case a portion only of the whole mass is engaged in the resistance of the whole pressure. Now percolation of the water into the body of the work places the levee under these very circumstances.

"A thread or plane of water finding its way into the interior of an embankment exerts just as much pressure against the earth on each side of it as if that thread or

plane were an ocean of the same depth as that thread or plane. As this thread separates the parts of the levee the outside water fills up the split, and by thus preserving the same height of water within the split as at the beginning of rupture, the levee becomes completely rent asunder, and thus reduced in its aggregate power of resistance is finally swept away. Porous materials, then, in water-banks, no matter what be their weight in the banks, tend by the insinuation of water-threads between their parts to the destruction of those banks—this tendency, however, being greatest at the time of the construction of the works, and least at the time when their adhesion shall have been perfected by the coating of deposit over their external faces, and the insinuation by filtration in their internal pores, of earthy matter.

"Loam is much better for water-banks than sand. Thirty per cent heavier, it meets all the conditions involved in leveeing on the ground of weight much better than sand. Much stancher in its parts, it is superior to sand in all those serious objections applying to sand for the purposes of water-tight embankments. The very best of those soils obtainable under the present practice on the Mississippi for the purpose of river banks is blue clay. Several kinds of this clay are found on the lines of the levee works, but they are all subject to the disadvantage of a greater or less admixture of fine sand. Perfectly impervious to water as they all are, the presence of sand lowers their usefulness partly by involving a lighter weight, but mainly, and sometimes even to a very serious extent, by giving them a tendency, especially after frosts, to melt or run like marl in water. But notwithstanding these drawbacks the clays of the Mississippi bottom furnish its very best material for leveeing."

There is much in the class of material employed, without doubt, but there is also need to place this material in a careful, proper manner. The best of materials will not give satisfactory results if thrown carelessly into an embankment, while an inferior grade of earth may be so built into a levee as to make a safe and permanent bank. Sound earth is a good enough material for levees, but the bank must be carefully built, of sufficient dimensions, and, especially with a light or treacherous subsoil, must have its base extended by a banquette.

The opinions of the Dutch engineers, who are probably the leading authorities of the world on the construction of levees, are worthy of attention in connection with this subject:* "The earth of which the dike is to be composed must be such as to cohere readily with itself and with the soil beneath it. The more cohesion the soil has, the more it is to be preferred; and the more will its different parts unite and form a compact mass which can oppose resistance to the water, and thus furnish a tighter dike. Clay is thus the most suitable earth for dikes, and for the most part is to be found along our coasts where dikes are to be built. It is to be procured, by preference, from the outer side, but when the fore-shore is scanty or wanting, it must be taken from the land side. Sand has very little coherency, and does not afford a water-tight and strong dike. Peat and swamp soil have too little specific gravity, often less

* Holland Dikes, Starling, Am. Soc. C. E., vol. xxvi., p. 694.

than water itself, and must thus, as well as sand, be rejected from the dike. Mould or arable land, though far inferior to clay, is still much better than peat or sand, packs closely, by reason of the smallness of its particles, and is especially suitable for dressing slopes that are to be sodded, as grass grows very well upon it. Clay cannot always be had in a pure state or in sufficient quantities, so that inferior earths must sometimes be mixed with it. But if the precaution be taken to work the best and purest clay on and near the outside, and the inferior sorts in the body of the dike, such sorts may be used without great danger. There are examples of dikes that consist of very sandy soil and have a covering of only one meter of clay on the outer slope, yet they furnish very satisfactory dams. It is easy to be seen, however, that such a dressing of clay must be treated with the utmost care, and the slightest injury to the outer slope must be immediately repaired, for if enough of the clay be removed to permit the water to come in contact with the sand or peat, very little confidence can be placed in the dike."

Construction.—As has been stated, great importance must be attached to the care with which embankments for water are built. If made simply, as those for railroads are constructed, they will be more or less permeable, and when the water comes against them settlement and deformation will result. It is necessary, then, that precautions be taken in cleaning up the foundation and in rolling or tamping the material in place so as to make it compact and close-grained.

Before beginning the construction of a levee it is important that all vegetable matter, trees, etc., be removed from the site. This should include the roots as well as the trunks and branches. After a thorough cleaning the ground should be plowed or spaded deeply in order to secure a more perfect bond with the new structure, and if the top soil is unsuitable it should be removed before bringing on new earth. Generally the specifications provide for cutting a muck-ditch near the center line of the levee, at the discretion of the engineer. This ditch and all excavations made in removing stumps, trees, etc., are filled up with approved material, well tamped.

The following clause from Government specifications gives the method of building:

"The embankment will be started full out to the side-stakes, and be carried regularly up to gross fill, in layers not exceeding 2 feet in thickness, when built by scrapers, and 6 inches when built by wheelbarrows. In wheelbarrow work the earth will be carefully tamped either by wheeling over the embankment or by employing one rammer to two wheelbarrows. When the embankment has been brought up to the proper height, it shall be dressed, and planted with living tufts of Bermuda grass, 4 inches square, and not more than 2 feet apart, well pressed into the earth and lightly covered with soil, to the satisfaction of the engineer in charge, or his designated agent. The contractor will cut down all trees, both great and small, to a distance of 100 feet from the base of the levee on both sides.

"Only clean, unfrozen earth, free from all foreign matter, shall be used in constructing the embankment. It will be procured on the river side generally. In no case must it be obtained within 40 feet of the base of the levee on the river side, or within

100 feet on the land side, and the side slope of the pit next to the embankment not to be steeper than 1 on 2. At intervals traverses must be left across the borrow-pits to prevent the flow of a current along the levee. The distances between the traverses will not be more than 500 feet. They shall be at least 10 feet wide on top, with slopes of 1 on 2. Borrow-pits must not exceed 3 feet in depth on the side next to the levee, but they may gradually deepen with a slope of 1 on 50 when on the river side, and 1 on 100 when on the land side of the levee. All existing levees, or parts of old levees, must be left, unless written permission of the engineer in charge is given for their removal."

In connection with methods of construction Mr. Starling says:* "Preference is generally given to the wheeled scrapers, especially if it is expected that water shall get against the new levee immediately. Generally, if it be possible, the levee is built a year or so earlier than it will probably be needed, in order to give it time to settle thoroughly, and to be completely covered with sod. With certain kinds of soil there is an objection to scraper-built levees, namely, that they are very liable to be cut and washed into gullies by rain before the sod has had time to grow. These soils are loam and mixed sand. When put up with wheelbarrows, banks of such material are at first comparatively loose and porous, and absorb water like a sponge. By the time they have settled fully, the sod has grown. When the earth, however, has been put up with scrapers it is very hard, and sheds water like the roof of a house. The material being light and friable, however, the rain soon cuts channels which it uses regularly, and the gullies which are the result of this action sometimes cut almost through the crown of the levee. A slope thus eroded has to be re-dressed before it is sodded, and the new dressing is liable to be washed away also. In spite of this objection scraper-built levees are generally preferred, and some engineers place such restrictions on wheelbarrow work as almost to prohibit it. The shrinkage exacted is generally one-fifth for wheelbarrow work untamped, or one-tenth if it be tamped, and one-tenth for scraper work."

Heretofore the construction of levees has been carried out almost entirely by hand methods, owing probably to the remoteness of the sites, and the difficulties attendant on the use of machinery in such work. Recently, however, steam dredges have been employed with success sufficient to demonstrate that by the use of suitable machines satisfactory construction can be obtained, and at a great reduction of cost.† To obtain economical results the dredges would be used as much as possible while the river was at bank stage, but would also be provided with pumps by which the borrow-pits, in which they were digging, could be flooded so as to keep them afloat. The wet material cannot be pyramided to the full height, and, in order to avoid loss by sloughing the dredge works several times over the line, adding to the material in place at each working after the material already put in has dried out.

Muck-ditch.—Mention has been made of a trench excavated along the line of the levee near the center of its base. This is called a muck-ditch, and it has a double pur-

---

* Levees of the Mississippi, p. 8.

† Annual Report Chief of Engineers, U. S. A., 1899, p. 3539.

pose in that it is an excellent way of investigating the character of foundation soil, and furnishes, when filled with selected earth and tamped, a diaphragm through which the water will not seep. This feature in levees has caused much discussion among engineers, some claiming that it is of little value, or is even hurtful, while others contend that it is a useful and even necessary precaution against destruction. Mr. Starling says: "Generally levee engineers, while they recognize that muck-ditches do a certain amount of good, as they have very limited means, prefer to spend their money above ground. They therefore give the ditches small dimensions, using them, in fact, rather for exploration than for any other purpose."

Mr. Hewson says: "The practice of cutting out a trench for the puddle, or 'muck-ditch' as it is called on the Mississippi, in the natural surface of the ground is generally useless, and sometimes positively mischievous. When retentive subsoils exist under the base of the proposed bank, then it is certainly a clear gain in stanchness to run down the puddle-wall of the levee to a bond with the underlying impervious earth. But the experience along the shores of the Mississippi leads to the presumption that, in those cases where the sand does not commence on the surface, a ditch of 3 feet deep is more likely to present a bottom of sand than of loam or clay. The rationale of those muck-ditches rests on their usefulness in preventing leakage; and therefore, supposing the ditch and wall carried up regularly with a puddle, those ditches in a great majority of cases failing to reach a more retentive soil than at the surface of the ground, involve in all those cases an utterly resultless waste of money. Besides, to undertake to prevent leakage through the porous earths of the natural shores of the river is a hopeless labor; and so far as the strength and durability of the levees are concerned is a labor also perfectly useless. It accomplishes nothing whatever for the artificial embankment. But in some cases these muck-ditches are, as already stated, mischievous. Across those lagoons or creeks which are dry during periods of low water the foundation for banks consists generally of a hard crust of clay for a few feet thick, overlying quicksands or thin puddles. These crusts, like the grillage of timbers used for the foundations of some engineering works, are highly valuable in those situations, by diffusing the weight of the superincumbent levee over a wide bearing; and thus, though unequally loaded by the necessary cross-section of the levee, assist, in proportion to their strength, in distributing that bearing equally. This, where not sufficient to obviate the sinkage altogether, reduces it considerably; and in bringing a large area to act in the resistance, assists in guaranteeing, with the least possible sinkage, and therefore the least possible loss of work and money—a finally well-sustained foundation. The muck-ditch, however, cuts this natural platform for the levee in two parts; and over this cut, the greatest weight, that at the crown, pressing vertically, acts as with a leverage in bending down, and finally breaking off, the natural crust of the surface. The necessity therefore follows, under those circumstances, of employing an excess of earth in forcing out laterally, and forcing down vertically, the running sand or soft puddle of the underlying foundation in order to compress those soft materials

into a compactness sufficient to present an effective resistance to the weight of the superincumbent embankment."

Banquettes.—As has been stated, in order to strengthen high levees a terrace is generally built along the land side. This is called a banquette. Its dimensions vary considerably in different localities, its crown at some places being about 20 feet in width, while at others it is twice that amount. It has a very flat slope and usually commences about 8 feet below the top of the levee, and is quite frequently used as a road.

Levee engineers claim that banquettes should be built at all points where the foundation or material of the levee is weak, and behind all embankments having a height of more than 12 feet, regardless of foundation or material, in order to have working-room and material in time of extreme high water. They should be built with the levee and not as a future enlargement, all later enlargements being made on the river side to cover weak spots in the old levee or its berme and to prevent sloughing.

In addition to strengthening the levee itself, the banquette is an excellent means of re-enforcing the natural ground underneath. This is frequently porous, light, and more or less traversed by roots, etc., and may give way to the pressure from without which will "blow up" the ground inside the levee. By building on this a bench of earth the hydraulic head is diminished.

The practice of using the banquette as a roadway is not a good one, but as those interested in levees would otherwise have to provide highways at considerable cost it is not practicable to wholly exclude travel. On some of those, where the crown width is about 40 feet, a space of 15 feet next the levee is fenced, the road being placed upon the remainder. In this way the levee is protected and the public accommodated. Some time ago the authorities in charge of levee-building decided to expend no money in improving embankments used as public highways, since which time a marked decrease in this use has been noted, and other roads are being built. In those localities, however, which are many, where it would be impracticable to get a new location within a reasonable cost, the levees are still used.

Although a roadway along a levee is without doubt injurious in that it allows stock, particularly hogs, to tear it up, and otherwise leads to its damage, there are yet some very good advantages to be derived from it. It facilitates general inspection and weed-cutting, and, in fact, all work pertaining to the construction and maintenance of the levee system. If travel be excluded in flood times there is not serious danger to the levee. Even without roadways hogs find their way onto the levees and do much damage.

Protection.—When a levee of any height has been completed it should be well sodded with tufts of Bermuda grass. This is the simplest and most useful of revetments; it throws out lateral runners and rapidly covers the surface, and grows well in exposed or sheltered situations. After it is once started it will withstand drought, freezing, and floods, and affords an excellent protection from rains, and even from wave wash, where the soil is strong. For a long time there was such strong prejudice

against it that its use was excluded, but now it is included in the levee specifications, being placed in tufts 4 inches square set 2 feet apart.

The growth of grass soon mats the covering together and makes a rather durable protection, and on the land side it is preferable to apply nothing else because it is important to be able to see the first signs of degradation. On the river slope at exposed points, flat stones (water-wings) are employed; but their expense is too great for the custom to be widely applied, and herbaceous or bushy vegetation is more general. Trees with high trunks and heavy roots must be excluded as they may become dangerous to the levee.

A recent Government report says that the high water "has demonstrated very clearly the value of a good sod which holds the slope in place, preventing sloughing even when the levee is saturated; also, the fact that 1 to 3 slopes, when well sodded, will withstand wave wash."

In cases of extreme exposure pile-work and broken stone are much employed, placed both parallel to the levee and in the form of spur-dikes. The protection of embankments by thickly growing grasses is very general in Europe. Willows and other forms of small trees are also planted for such purposes on the river slope, thus holding the earth by their roots and breaking the wave-current by their branches. In some countries straw is used, twisted into strong ropes, which are laid along the levee side by side and held down by stakes.

Maintenance.—The maintenance of a levee includes all measures necessary for its protection and welfare, not only when it is threatened but at all seasons. The cutting of weeds and repairs of injuries by stock are as important as those of greater magnitude during flood times. The measures for high-water protection are often, of necessity, crude and of a temporary character. Usually they are carried out under the direction of proper authority, but there are times when the population must take matters into their own hands and hastily improvise such works as will protect their interests.

Experience has shown that when levees break it is ordinarily from one of the following causes:

(1) Insufficient height.
(2) Leakage.
(3) Sloughing.
(4) Wave wash.
(5) Cutting.

(1) It is evident that a levee which is low enough to permit the water to flow over its crown will, under ordinary conditions, be destroyed unless protection is promptly given. At first the water merely trickles down the embankment, causing a slight wash; but this rapidly enlarges as the flow increases, until, finally, the waterfall pours through the cut and tears out large masses of earth. Once well started it is difficult, if not impossible, to stop it, so that it is far better to apply the remedy in advance of the real danger.

The prevention of a break from this cause consists in raising the grade of the embankment by means of earth taken from the most convenient point and placed upon the crown. Too often this point is the body of the levee or its banquette, because everything else is under water at the time. Much of this raising of height is done with teams and scrapers, but as the addition grows in height it also decreases in width, so that it may be necessary to complete it with wheelbarrows. Another common method of increasing the height is to set posts or drive stakes into the front edge of the crown and against these set up planks. This structure is then backed up with earth. Probably this is the least expensive and quickest method of increasing levee height. Another method largely resorted to is the placing of sacks filled with earth on the top of the levee, sometimes in single tiers, sometimes piled to considerable height. They are backed up with earth if considered necessary.

(2) Leakage may arise from several causes, but one of the most common is the crayfish. Burrowing animals of several kinds also work holes through or under a levee, and once the water is started through these openings it rapidly cuts away the embankment until it is checked. It is not probable, however, that all holes can be ascribed to these causes, and part of them at least are chargeable to the decay of roots in the ground. Whatever be their origin it is necessary to look after them promptly when they begin to discharge muddy water, for then it is evident that erosion is taking place within the body of the embankment, and that the opening is being enlarged.

Even when levees are perfect in themselves there is often a continual transpiration of water through the foundation soil during flood stages. This water, called "seep-water," is not only injurious to the lands within the inclosed basin, but may also become dangerous to the stability of the levee, should the base be somewhat narrow. Hence the necessity for the banquette. It is not unusual to see the water break out through the soil inside the levee and throw up considerable mounds of sand. These eruptions are locally known as sand-boils, and as a general thing cease to throw out anything but water after a short time. However, during a flood all these leaks, of whatever nature or origin, may become quite dangerous, and then it is necessary to arrest their flow. The method in most common use for this purpose, both in this country and abroad, is to surround them with a small levee, the ends of which are joined to the main embankment. This is called "hooping." This little bank is usually built to a height about 2 feet below the river level, thus relieving the levee of a portion of the strain and reducing the erosive action within.

In regard to "sand-boils," Mr. Starling says:* "Sand-boils are very common and very alarming incidents of every high water, and have been especially prevalent since the great accessions which have been made in recent years to the height of levees. At their first occurrence a stream of water suddenly bursts through the ground, throwing out volumes of sand of several cubic feet, or perhaps even of yards, which it distributes around the circumference of the hole. Sometimes large numbers of these outbursts

* Levees of Mississippi, p. 13.

occur in the same locality; indeed there are some situations which are peculiarly liable to them. Twenty or thirty will be found in a length of 200 or 300 feet. They do not usually occur very near the base of the levee, but 40 or 50, 100 or 200 feet away; indeed they are occasionally found 1000 feet distant from the levee. . . When the boil is near the levee and of considerable dimensions, it is unquestionably a symptom of danger. Perhaps a little more, and it would have been a crevasse. . . . When the manifestation is formidable, the remedy generally applied is to weight down the weak place with an additional load of earth, which is usually laid upon a thick layer of brush, to allow a partial drainage of the leak water and to prevent the contamination of the new and dry earth by the mud of the old."

(3) One of the most alarming experiences of the levee engineer is that which comes when, with the water rising upon the outside to nearly the full height of the levee, he sees the land slope sloughing away from the excess of permeability of the embankment. The chief cause of sloughing may be laid to a lack of proper care in construction. Had the earth, during the building of the levee, been well tamped in layers, saturation would have been less liable to occur, and their rupture might have been avoided. A French authority* has thus described the process:

"In the degree that the water rises in the river, oozings of increasing extent are noticed on the opposite side of the levee, and where the maximum of permeability exists (taking account of the pressure and thickness), the slope on the inner side is weakened and a first subsidence is produced. This subsidence, often small in itself, is followed by others which come more rapidly as the permeability is increased by the diminution of the thickness. Then, after a continuance of these slips, the rest of the bank, having no longer a sufficient resistance, is carried away and the break enlarges rapidly. It is seen that the rupture does not occur suddenly under the influence of saturation; the dike melts and is liquefied like a morsel of sugar in contact with water; it spreads out on the side exposed to the air and only gives way finally when these seepages have sufficiently weakened it. The damage may, in fact, be attributed to a double cause; firstly, that the slope which too light earth takes when dry is not sufficient for equilibrium when it is wet; secondly, that this earth is carried away by infiltration."

In order to prevent this seepage and sloughing several solutions have been offered and tried. In France, a stanch revetment of stone paving has been placed along the river side of the levee to prevent contact with the water. This is very expensive, and has led to much criticism, not only on account of the cost but also because, it is claimed, breaks in the paving will occur and allow the water to come against the bank, thus destroying its effect. Another solution lies in giving the inner face the slope which its material would take when wet, but as this does not prevent the removal of material by infiltration it has been proposed to wall this inner slope with dry stone in order to

* Rivers with Free Current, De Mas.

prevent this escapement. This has been done in the protection of cities, but is too expensive to secure general application.

The high waters of recent years in the Mississippi valley have demonstrated pretty thoroughly the necessity for a complete drainage of levees in order to overcome this difficulty. Ditches of sufficient capacity to carry away all seep-water are cut along the base of the embankments, and little ditches are made down the slope into them. The latter lead the waters off as rapidly as they percolate through the pores. This prevents the softening of the material and the sliding which would otherwise take place. This practice is becoming pretty general and has proved quite effective, judging by the reports of engineers who have resorted to it. When the sloughing has been started it may be checked by driving heavy stakes well into the embankment and bracing them back to similar stakes. In front of the first row of stakes, brush, saplings, or any convenient wood must be laid horizontally to form a barrier for the earth to lodge against. Brush is also placed upon the portion sloughed and covered with earth until the normal levee section is practically restored.

Another method of the treatment of sloughing levees is quoted from a Government report: "Treatment of sloughs is simple if the principle involved be understood. It is usually the case if one occurs where an inexperienced man is at work, that he will endeavor to restore the lost section with earth or sacks. This does no good, but rather tends to augment the trouble, because it makes it more difficult to drain the slough and at the same time puts that much more weight on the semifluid mass to squash it out; squashing the sloughed mass out invariably pulls with it some of the standing sections. He does this because he does not understand what produces the slough. He does not know it is caused by too free leakage through the embankment, and insufficient land-side drainage.

"It is a well-known fact that if any vessel encountering hydrostatic pressure be leaking, the leak can be more expeditiously stopped from the outer or pressure side than from the inner side, provided, of course, the opportunities for reaching both are equally good. If a barge, for example, loaded with coal starts a leak in a side seam, it is impracticable to dig away the coal to get at it. It is equally impracticable to detect the leak by feeling along the outer side of the barge as the inflow is too light to be detected by hand. But the deckhand can take a supply of coarse sawdust, and, by means of a long handle, can lower a cup of sawdust into the water in close proximity to the leak, shaking it gently next to the barge, and as the sawdust floats out of the cup some of it is drawn into the crack and becomes lodged; it very shortly swells and the leak is choked. Just so it is with seeping levees, but their treatment consists in dumping loose earth in the water over the river slope. The particles of earth are drawn into the interstices, become swollen, and shortly the leak is choked. The layman who sees this work in progress will immediately classify dumping loose earth in the water as sheer nonsense and not possible of practical results, but as a matter of fact, in a short while after work has been commenced, the good effects will be shown by reduced

seepage, and a little later by its entire stoppage, followed by no further inclination of the mass to move, but rather by a commencing to dry out. Of course this treatment does not apply to crayfish or other leaks where there is a channel of any considerable size. If sloughs be treated when they first occur, or, better still, before they occur, when their approaching occurrence is clearly indicated to the experienced eye by the accumulation of moisture on the land slope, much annoyance and expense can be avoided."

(4) Wave wash has been mentioned as one of the causes of levee failure. It attacks the river face of levees, particularly new ones, owing to the wind or the passage of boats. When trees intervene between the levee and the river its effect is not felt, or if it is, it is not usually disastrous.

Where winds continue with considerable strength for several days, as is often the case during the spring, it is no easy task to maintain the levees. The waves roll at considerable heights and strike with force, and it requires good material to withstand their action. The following methods have been tried for the prevention of wave wash: 1. Placing a continuous strip of bagging or burlaps along the zone of the levee slope affected by the waves and securing it by pegging it to the ground; 2. placing sheets of corrugated iron, overlapping longitudinally; 3. building a bulkhead of boards along the front slope; 4. securing a floating boom, composed of logs chained together, along the levee.

The most general method used, however, is to protect the part of the slope affected with bags filled with earth, placed something after the manner of shingling a roof. Each of these methods is very expensive and has its objectionable features, and where the extent of the damage is not likely to prove a menace to the actual safety of the levee it would probably be better to make no expenditures for protection against wave wash, as it might be cheaper to replace the material washed away after the water has subsided.

The work of restoring the slope and resodding should be done as soon after the flood as practicable, in order to allow the new material all the time possible to settle and the new sod to have all the advantage of the early season to get well set and obtain a good growth before another flood. Much good could be done by cultivating a growth of willows and cottonwood along such exposed fronts to break the force of the waves. Both of these plants are very hardy and have a rapid growth, and it is probable that plowing a few furrows and covering up live portions of willows and cottonwood would be sufficient to build up a living breakwater which would be very effective and inexpensive in future levee protection.

(5) Cutting is the fifth and last cause given for the failure of levees. This is rare, although through maliciousness, deranged mind, or other reasons levees have sometimes been cut. It is virtually impossible to guard against it unless the systems are placed under strict police surveillance, and this is not practicable.

Regarding levee failure an authority states that: "In spite of all precautions a crevasse will sometimes occur. It is generally because of some hidden defect which

could not be suspected, or some circumstance that could not be seen or controlled. Levees very seldom break from sloughing, for that is generally remedied. They seldom, of late years, break from insufficient cross-section; they never break from sliding or overturning, as they ought to do by rule. They do not often break from storms, though sometimes they have run very narrow escapes. Crevasses are due mostly to three causes: to being overtopped by the water, to holes through the levees, and to weakness of the foundation. In former years the most common of all causes of breaks was insufficiency of height, because even a thin stream of water running over a bank of light or ordinary material will effect its destruction. Experience having taught that this was the chief danger to be apprehended, construction has been generally turned in the direction of increased height.

"The closing of a levee break, or crevasse, as it is generally called, is an operation which ranges, in point of difficulty, from the easy to the impossible, depending upon the height of the levee, its material, the foundation, and a number of things. The only plan which has been successful is one of the oldest employed. First, a number of bents or trestles are set up, and connected by stringers; then hand-piles are driven in front of the stringers, supported by the latter. The work is completed by filling in with sacks and earth.

"Another method consists of throwing out spur-dikes at right angles to the break and thus destroying the effect of the current, while yet another surrounds the break with a pile-dike, which is then rapidly filled with sacks of earth. As may be imagined, earth is scarce at times of floods, and this fact will often defeat the closing of a crevasse which might otherwise be successfully stopped." *

Drainage of Inner Basin.—The drainage problem of the areas inclosed along the Mississippi is a much easier one than in many other sections where levees are employed. The continuous slope of the flood-plane and its magnitude render great service in this regard. Each system of levees incloses an area which is drained by rivers, creeks, branches, etc., reaching every portion of the basin and finally emptying into the main river itself. The highest land to be found is on the bank of the river, from which point it gradually slopes back to the drainage tributary, which is generally near the hill ground. Thus it will be seen that there is a complete system of drainage for each of the various levee basins through the natural stream traversing the inclosed areas, the final outlets of which are not closed by levees because, if this were done, it would of course be necessary to exhaust the water from the basin by pumping. On account of its great quantity this would be impracticable, to say nothing of the expense, which would not be justified by the value of the lands inclosed.

By starting at the hills at the head of a basin and carrying the levee along its front as far as the outlet of the drainage tributary at the foot of the basin, it will be seen that water in entering must either pass through or under the levee, or come in through the tributary stream. That which passes through and under the embankment, while of

* Levees of the Mississippi, p. 14.

serious import to the stability and permanence of the levee itself, may be carried into the nearest branch of the drainage system without difficulty, because its quantity at any one point is not great; but that which enters through the gap formed by the outfall may be considerable, and if it were not for the slope of the flood-plane already mentioned, the entire basin would be inundated. To more fully illustrate this we will quote Mr. Starling:* "If the front of a basin be sealed by levees, joined to the hills at its head and extending as far as the mouth of its drainage stream at its foot, the water can get access to it only through the gap caused by the entrance of the tributary. The plane of the Yazoo basin has a mean slope of about 8 inches to the mile. At a distance of 15 miles above the lower end of the levee system, therefore, the level of the back-water from the Mississippi will be about 10 feet below that of the river-water. Now, the hills which bound the basins usually approach the river gradually, so that the lower end of the basin has much less than the average width. In the instance cited the levees extended to the mouth of the Yazoo River. The area of the alluvial tract from its mouth to a line 15 miles above it would be about 250 square miles. In the case of the St. Francis it would hardly be more than half of this amount. Of these areas large tracts have so high a situation that they are several feet above back-water. Part of the remainder is irreclaimable swamp."

The question of drainage is, by reason of physical features, not so readily solved in many cases as in that of the Mississippi, and recourse must be had to pumping and other devices. The operation is a difficult and expensive one, and involves the creation of ditches and drains in the lowest levels, with works for the regulation of flow at their outer or lower extremities. These works must not only provide for outward flow but must prevent, as far as practicable, that which would come inward. During periods of low water these regulating gates remain open, but upon the approach of a flood they have to be closed, and the waters within the levee must remain and gradually rise over the lands unless removed by pumping. Whether this can be done economically depends upon a number of conditions, among which is whether the value of property, crops, etc., which may be injured by reason of this elevation of the water-surface, will justify the expense necessary to preserve it.

Influence on Flood Heights.—As levee systems are developed and narrow the field of inundation, it becomes necessary to raise their heights, because the elevation of high water is increased by reason of this restriction. The Mississippi systems are not yet complete, but sufficient progress has been made to indicate decidedly that the flood level has been raised materially, and that the completion of the various systems will be the signal for greater flood levels. It will then be a struggle to hold the river within certain prescribed limits.

Experience in other countries has shown that in this struggle to keep rivers within restricted bounds they often regain their domains, and it becomes necessary to rebuild, strengthen, and increase heights of levees.

* Levees of the Mississippi, p. 1.

Each extension made is the signal for a further rise of elevation. An instance in point is given by De Mas:* "From the commencement of the eighteenth century the maximum heights given by a gauge near Ferrara have followed an upward march. Before 1729 the highest figures remained under 23 feet; from 1729 to 1809, or during a period of eighty years, the 23-foot mark was often exceeded without the 26-foot mark being reached. Since 1810, that is, during a period equal to the preceding, the 26-foot mark was passed five times."

Many years ago a French engineer stated that the high-water level of the Po had increased 6½ feet in two centuries, and that, while the number of breaks in the dikes was only 41 in the eighteenth century it had been 119 in the first seventy-two years of the nineteenth, 36 of which were in the year of 1872 alone.

On the Theiss, the embankment works of which are not less in importance or in results than those of the Po, there has been a steadily increasing elevation in the heights of floods. These increased levels have not come without bringing disaster. In 1879 the city of Szegedin, one of the most important of Hungary, and having a population of 75,000, was almost destroyed by inundation. The levees were repaired, rebuilt, and increased in height so that even higher floods have since passed in safety. In alluding to this the authority just quoted asks: "Will it always be thus? Will not the height of floods continue to increase in a manner dangerous even to dikes thus strengthened? Will not the old river-beds, kept open for the escape of great floods at the location of each of the cut-offs, silt up little by little? Can the surveillance never be relaxed which was made necessary by a disaster still fresh in the minds of all? When we see what happens elsewhere we cannot fail to regard this matter with apprehension, and in rendering due credit to this remarkable work we ought not to overlook the possibilities of the future."

A similar experience is recorded with the levees of the Loire. The crown, which was originally placed 15 feet above low water, was raised to 21 feet after the flood of 1706, and even this has been found to be too low, for all the great floods have continued to rise and to surmount the embankments. After the extreme high water of 1846, an additional height of over 3 feet was placed on the levees, but the floods of 1856 and 1866 demonstrated that this raising was again insufficient.

The argument is frequently made, and is almost universally believed, that by confining a river at flood between levees increased scour of the bed and banks will take place and thus give a greater section for discharge and a greater velocity, the result of which will be to reduce not only the elevation of low water but also that of floods. It is quite probable that a reduction of the height of low water will follow the establishment of levees, but this does not necessarily indicate that the high-water line will also be lowered; in fact, it is necessary to give up this doctrine if we are to profit by experience, because each successive exclusion of territory on the Mississippi has been followed by a higher flood-line. In referring to the flood of 1897 Mr. Starling says:

* Rivers with Free Current, Art. 178.

"Its principal interest to the engineer is due to the experience which has been derived from the wholesale closure of unleveed tracts and the extraordinary elevation of its high-water line consequent thereon.

"There are two of the great basins into which the Mississippi valley is divided which have only recently been protected to any extent by levees. These are the St. Francis and the White River basins. The former was closed during the last three years, or since the flood of 1893, to a distance, measured along the river, of about 120 miles. There still remains a gap of about 100 miles. The White River basin has been undergoing a gradual process of closure for several years. In 1893 there was a gap of about 15 miles, extending between points 330 and 360 miles, respectively, by river, below Cairo. In 1896 this gap was closed and the line of levee was made continuous from the hills at Helena to a point 8 miles above the mouth of White River.

"It is to the building of these lines and to the maintenance of the lines previously existing until a late period of the flood that the unparalleled stages attained by the water have been due."

Efficiency.—The engineer Belgrand has voiced his experience in regard to levees as follows: "In my opinion it is plain that even in a country where levees have existed for twenty centuries, where property has been exposed to all the consequences—I refer to the valley of the Po—it has not been clearly demonstrated that the advantages are greater than the inconveniences."

Whether we must eventually come to this conclusion as to the Mississippi levees time alone can decide. Without doubt there are serious objections to them, and it is probable these objections will not decrease in number or strength after the virtual completion of the systems. To quote from one of the authors just mentioned: "The best study they (levee engineers) can give to the subject—and some of them have given a great deal—lead them to think that levees are the most efficient, cheapest, and most certain means of securing the lowlands from inundation, while preserving existing conditions and improving rather than deteriorating the channel. Very considerable progress has been made toward the completion of a levee system, and it may be said that the end is plainly in sight. The engineers, then, and others charged with the responsibility of protecting the lowlands deprecate the dissipation of the none-too-plentiful funds in experiments which will certainly be costly, and which they believe will be unsatisfactory or actively harmful, when a plain road to safety lies before them. Half of this road has been traveled. It is not short at best; but it may be made indefinitely longer if we stray into every by-path that presents itself.

"The objections which have usually been urged against levees are: first, that they are too precarious—that they cannot be made strong enough to be secure against breaking; second, that they cost too much; and third, that they raise the bed of the river by confining within the channel the silt which otherwise would be carried over the banks and be deposited on the adjacent bottom lands.

"The first objection will not be entertained by any engineer when he hears that

the highest banks do not exceed 40 feet, and that this height is only for a few hundred feet at most. It is, then, practicable to make the few high levees of any dimensions that may be necessary for safety with slopes of 1 to 10—if less will not do—without inordinate expense. No such proportions have ever been found necessary. About 1 to 5 is the flattest slope that has ever been used. It is not the great levees that break.

"Even when ultimate grade is attained, say 3 feet above the 'potential high water' of 1897, the average height of the levees will not exceed 18 feet, of which 3 feet will be a margin against storms or accidental deficiencies, settling, etc., leaving a water-head of 15 feet. Extreme high water will last only a few days; within a foot of extreme height it may last three or four weeks. For such embankments, in ordinary soils, slopes of 1 to 3, with banquettes, will be sufficient. . . . As to cost, it may be said that levees are the least expensive means of reclaiming overflowed lands that have ever been proposed. . . . The idea that confining a river should cause it to deposit silt is so contrary to reason that it is a wonder it ever obtained credence at all. Transporting power is generally believed to be proportional to the square of the velocity. The confinement of the stream unquestionably gives increased current. Undoubtedly, in retaining the water within the channel we also retain the silt, but at the same time we retain the vehicle by which to carry it, and give the vehicle greater capacity.

"General Comstock's conclusion was that in the cases of the Po and the Rhine the rise of bed, if any, was insignificant, and that in the case of the Mississippi there is no evidence of any at all."

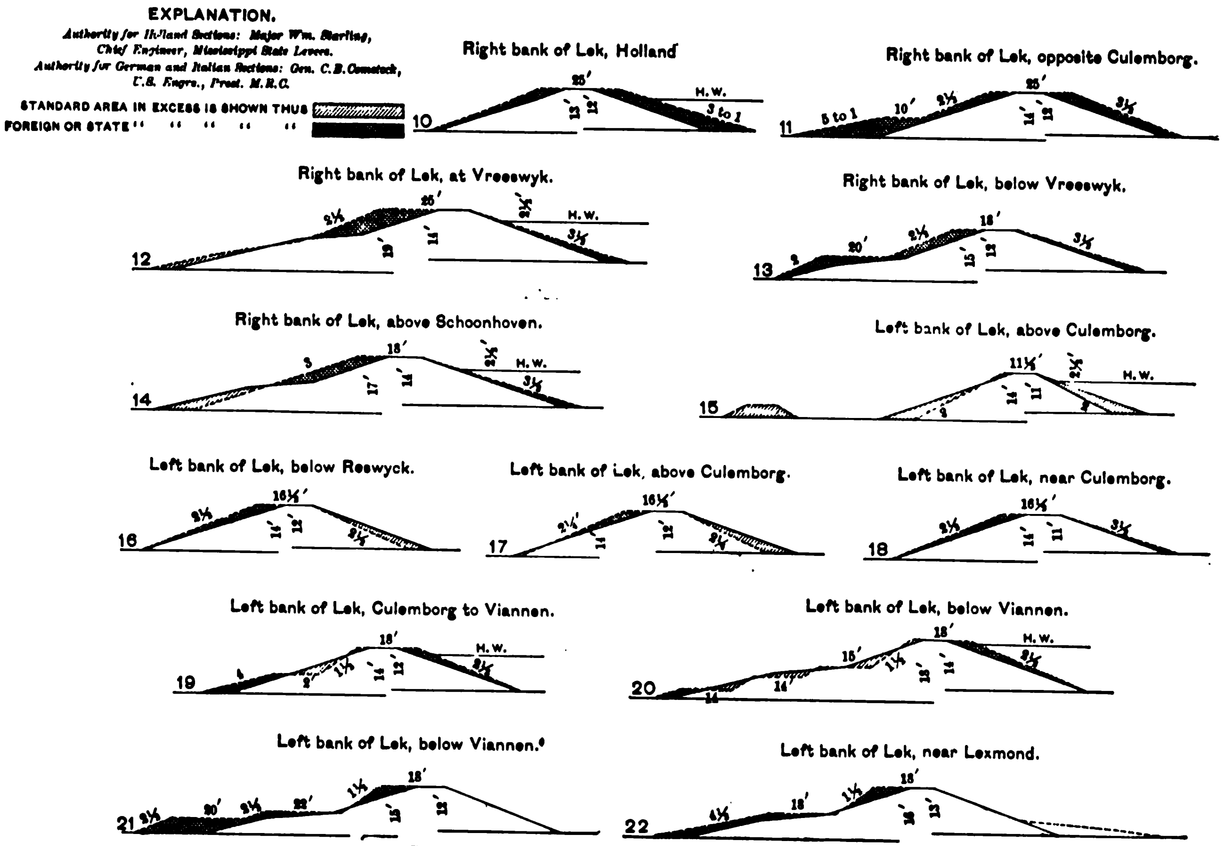
Cost.—A recent Government report shows that the United States and the States of Arkansas, Missouri, Mississippi, and Louisiana have expended on the Mississippi levees the sum of $39,600,000 during the past eighteen years. A levee engineer estimates that their total cost has been over $50,000,000, of which the United States has paid over $15,000,000. This is about $35,000 per mile.

The report gives the following:

Miles of levees in service.	1,436.3
Contents, cubic yards.	150,595,000
Required to complete the system, cubic yards.	109,090,000
Annual loss by crevasses, caving, and other causes.	2¼ per cent.

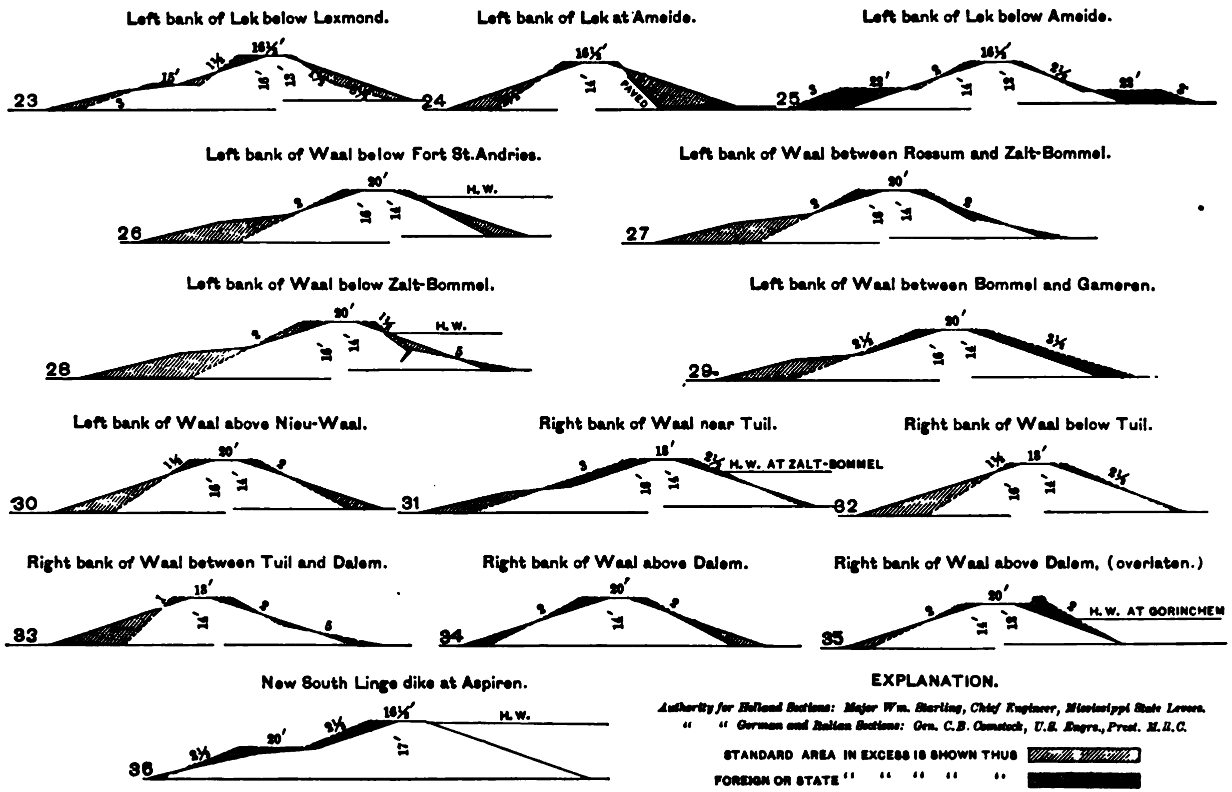
Foreign Levees.—Mention has been made of the levees along the Po, the Theiss, and the Loire, and owing to the extent and age of these works further information in regard to them will be of interest.

River Po.—Usually the levees on the Po are located at a considerable distance from the banks, but occasionally they come quite close, and at such places are protected by revetments or fascines, an inner levee being frequently built behind them for fear of accident. They extend up each side of each tributary to such distances as floods are to be expected. In a word, the great submersible plain of the Po is guarded by a network of embankments enveloping each affluent and laying out for each a minor bed which must be followed by all the water falling in the valley. These levees are



SECTIONS OF LEVEES IN HOLLAND.

(To face p. 98.)



SECTIONS OF FOREIGN LEVEES.

(To face p. 99.)

located wider apart in the neighborhood of tributaries of great flow in order to provide a greater area for the increased discharge, thus forming reservoirs in which is gathered not only the surplus water, but tremendous quantities of sediment as well. In regard to the result of this method De Mas says:* "The controlling levees have been spaced so that in the localities where affluents abound the major bed forms a sort of reservoir, in which are stored not only the floods, but also the deposits carried by turbid waters. It follows from this arrangement that the major beds of the Po and of its affluents in their lower parts serve as regulators to each flood descending from the mountains; and after having distributed it they lead it to the sea through a contracted outlet which modifies the velocity and the disastrous effects caused by the raising of the flood-level above Panaro. This distribution was first brought to attention when Lombardini stated two remarkable facts which are shown also in the report of the engineer Baumgarten. The first is that the discharge of a flood is very nearly the same at Tessin, at Cremona, and in the vicinity of Ferrara. The second is that while all its affluents together discharge 528,000 cubic feet per second, the discharge of the Po is about 176,000 feet for the same unit of time. The greatest part of this remarkable result should be attributed to natural circumstances. It is certain that the affluents from the Apennines discharge before those from the Alps; and there is no doubt that the lakes Maggiore, Como, Garda, and others retard the most powerful torrents. Thus it was found that at Lake Como the maximum discharge in the flood of September, 1829, was at the entrance 68,501 cubic feet per second, and only 28,389 cubic feet at the exit. It will be seen, therefore, that the flood was diminished in passing through the lake in the ratio of 2.4 to 1. But it is equally certain that in the establishment of levees whose effects are to produce certain contrary results natural laws must not be neglected, and a knowledge of these must govern us accordingly. Thus, according to the same experienced engineer, the volume stored between the levees of the Po and its affluents, from Casale to the sea, is 66,739,000,000 cubic feet, which corresponds to more than four days' discharge of the river at the rate of 181,280 cubic feet per second. In reality the hand of man has created a vast regulating reservoir which affects the régime of the river in the same way that the lakes cited above affect its affluents."

The Theiss.†—The Theiss flows through the plain of Hungary, which in general consists of a layer of vegetable earth overlying a thick stratum of compact black clay. The former constitutes the banks and the latter the bed of the stream. Like most rivers the slope is great in the mountainous portion, diminishing continuously in the plain. The heights of the floods increase as the slope decreases. They come in the spring, and are caused by the melting of snow in the Carpathian Mountains. They are slow, often taking several weeks in which to reach their full height, and their duration at the highest level is generally several days. The maximum discharge at Szegedin is given as 123,200 cubic feet per second. The river always maintains a splendid navigable depth.

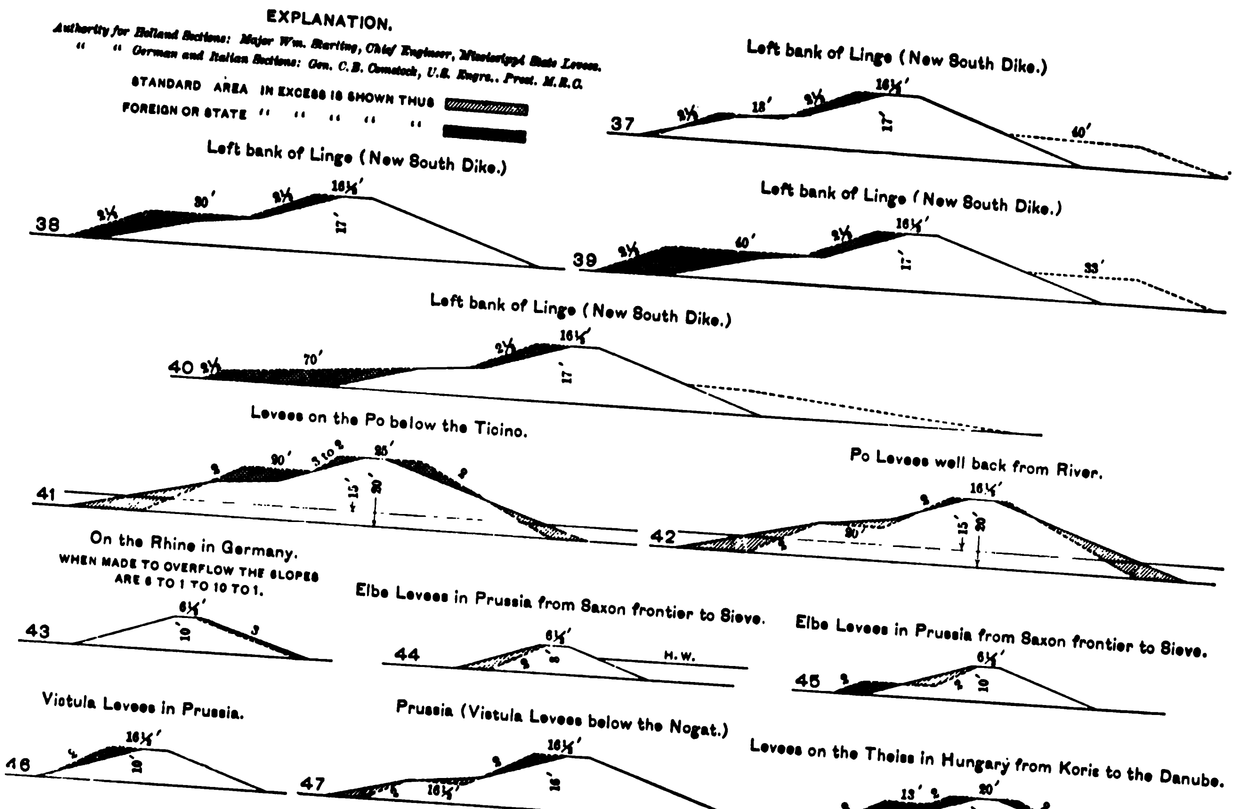
* Rivières à Courant Libre.

† Annales des Ponts et Chaussées, 1890.

Levees continue without interruption along the Theiss and its principal tributaries. Their entire surface is sodded, but the sod is not always proof against floods, and it is found necessary to resort to other means of protection, such as fascines, planting of willows, etc. These embankments are usually at some distance from the river bank, generally from one-half to three-quarters of a mile where the river is 600 to 800 feet in width. They are never built within less than one-quarter of a mile of the river-bank, and are often widened by cut-offs. Although of recent construction they are quite as important as those of the Po, and the results have been equally satisfactory. Their maintenance is largely in the hands of the adjacent landowners.

Loire.—The levees on the river Loire are usually built upon one side of the stream only, the opposite side being a slope or a bluff. This arrangement is occasionally varied where the river crosses from one side of the valley to the other. The discharge below Bec d'Allier is about 352,000 cubic feet per second in the greatest floods. This great quantity of water is concentrated in a narrow bed, the width of which was not fixed by a consideration of all the necessities of the case. The addition of new levees has complicated the matter, and it is stated that the crowns have been raised 9 feet within two hundred years.

Few of the tributary valleys are completely embanked. At the upper end levees have been built over a greater or less length. The width of the river in high water is quite variable, varying from 2000 to 7000 feet.



Sections of Foreign Levees.

To face p. 100.)

CHAPTER VI.

STORAGE RESERVOIRS.

General.—Nature has indicated one satisfactory method of improving the navigability of watercourses, in the lakes which lie at the foot of mountainous regions and from which rivers flow. By them the length of the navigable season is increased and the danger from floods is decreased, and the lesson taught is that where artificial lakes or reservoirs can be constructed near the sources of streams, the waters falling in the various basins leading to these reservoirs may be usefully stored up. Not only will excess of water be thus held back while that entering lower down is making its escape, thus preventing a flood, but it may be drawn out as required by the necessities of navigation and to its great benefit.

About the year 1800 Thomas Telford, a distinguished civil engineer of England, wrote a work advocating the storage of flood-waters and urging its adoption for the improvement of the navigation of the river Severn. His idea was "to collect the flood-waters into reservoirs, the principal ones to be formed in the hills of Montgomeryshire, and the inferior ones in such convenient places as might be found in the dingles and along the river. By this means the impetuosity of the floods might be greatly lessened, and a sufficient quantity of water preserved to regulate the navigation in dry seasons, etc. This, it is thought, might now prove the simplest and least expensive mode of regulating navigable rivers, especially such as are immediately on the borders of hilly countries." Another English engineer, William Jessup, also gave the matter considerable thought, and expressed the opinion that "rivers may be rendered nearly uniform throughout the year by reservoirs." Mr. Rennie, however, also an English engineer of distinction, ridiculed the ideas of Telford and Jessup in regard to the correction of floods by such means.

Charles Ellet, Jr., and Elwood Morris, both well-known engineers of their day, strenuously advocated the reservoir plan for the Ohio River. In 1857, however, W. Milnor Roberts, one of the ablest authorities on river improvement this country has had, carefully investigated the plan and made the following statement: "My own careful investigation of the subject of controlling the floods of the Ohio by means of artificial reservoirs satisfied my mind conclusively that such control by any human means attainable within the practicable limits of cost is impossible." Mr. Roberts gave his views in the Journal of the Franklin Institute in 1857. He proved from an examination of the records of the floods on the upper part of the Ohio, that some of the highest

floods occurred when such reservoirs, had they been in existence, would have been full. Such being the case, they could not have materially aided in restraining those floods, and this would certainly be the case almost every year owing to the irregularity of the periods when great floods occur.

"If by possibility there could be a gigantic dam 400 feet high at Wheeling, sufficient actually to stop and absolutely to control all the water of the 27,337 square miles of drainage above Wheeling, it could not restrain any portion of the flow from the remaining 189,663 square miles of the Ohio valley, nearly seven times the area. We should even then have control of only about one-ninth of the Ohio River territory. As a practical engineer I cannot hesitate, therefore, in expressing the opinion, that the scheme of controlling or equalizing the floods of the Ohio River by means of artificial reservoirs is certainly impracticable; and that in any merely human view of the question it is practically an engineering impossibility."

This reasoning is applicable to many other cases as well as to that of thė Ohio.

After the inundations which devastated France in 1846, 1856, and 1866, the question of reservoirs was widely discussed, as mentioned farther on, but their excessive cost prevented their application on a great scale, and a French authority has in recent years stated that "the idea of modifying immediately the régime of inundations by the creation of a system of reservoirs is now considered as unrealizable."

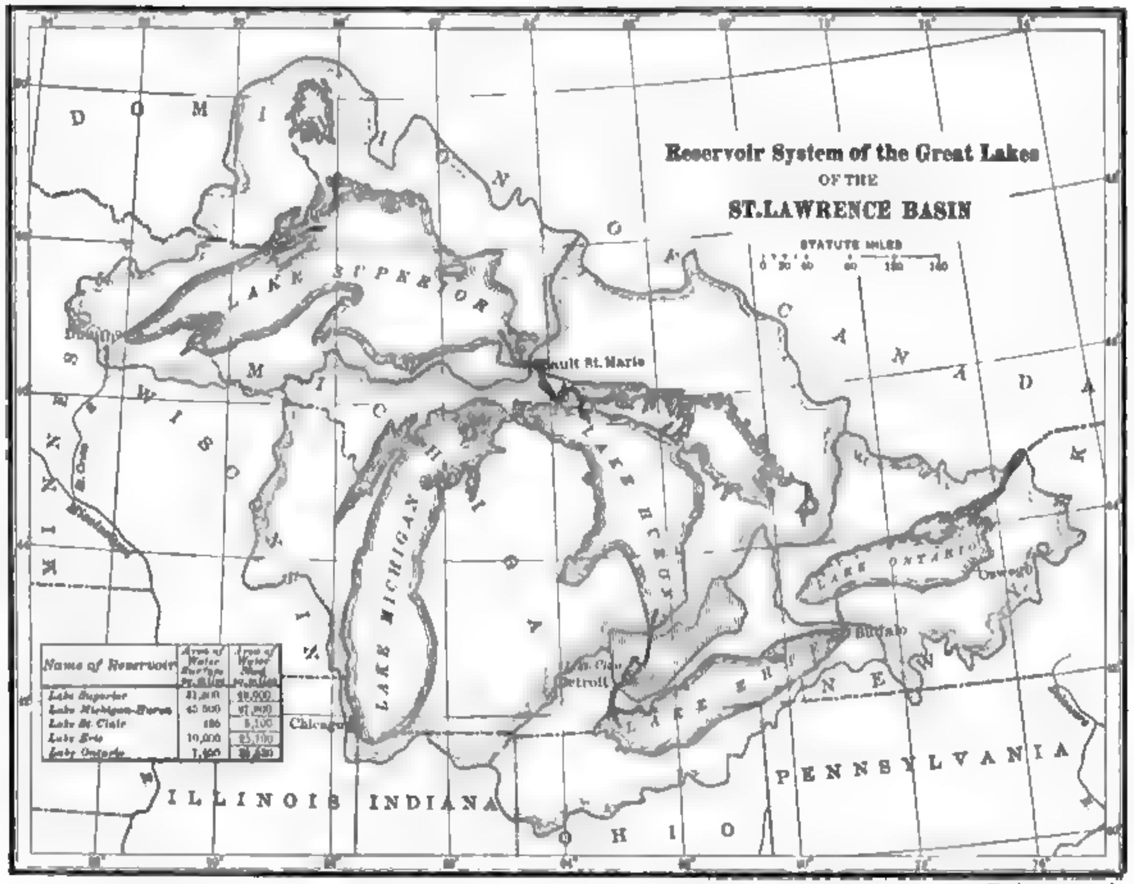
The question of storage reservoirs has been exhaustively entered into by a recent Government report, from which the following extracts have been made:*

Natural Reservoirs.—"Nature presents abundant examples of the effective control of stream-flow through the agency of reservoirs. There are indeed comparatively few streams whose flow is wholly uninfluenced by such action. The most perfect example in the world, both as to the magnitude of the stream and the completeness of control, is the St. Lawrence River, embracing the great chain of North American lakes. Considering only that portion of the system which lies above the Falls of Niagara, let the flow at the outlet be compared with that of other streams of similar magnitude. For this purpose take the Niagara River at Buffalo, the Ohio at Paducah, Ky., the Missouri at its mouth, and the Mississippi just above the mouth of the Missouri. The following table gives the area of watershed in square miles and the mean annual discharge in cubic feet per second of each:

	Niagara.	Ohio.	Missouri.	Mississippi.
Watershed.......	265,095	205,750	530,810	171,570
Discharge.......	232,800	307,000	100,000	130,000

"The above discharge for the Niagara River is based upon twenty-five years' record (1871–1895); that for the Ohio and Upper Mississippi upon six years' record (1880–1885); and that for the Missouri upon twelve years' record (1879–1890).

* Reservoir Sites in Wyoming and Colorado, Captain Hiram S. Chittenden, Corps of Engineers, U. S. A, House Doc. 141, 55th Congress, 2d Session. 1898.


Reservoir System of the Great Lakes
OF THE
ST. LAWRENCE BASIN
STATUTE MILES
Name of Reservoir
Lake St. Clair
10,000
LAKE SUPERIOR
LAKE MICHIGAN
LAKE HURON
LAKE ERIE
LAKE ONTARIO
Sault St. Marie
Chicago
Detroit
Buffalo
Oswego
DOMINION OF CANADA
MICHIGAN
WISCONSIN
ILLINOIS
INDIANA
OHIO
PENNSYLVANIA
NEW YORK

To face p. 102.)

"The maximum and minimum discharges, except for Niagara, show a much greater divergence, the ratios of $\frac{\text{maximum discharge}}{\text{minimum discharge}}$ for 1883 being as follows:

"Niagara, 1.19; Ohio, 28.22; Missouri, 29; and Upper Mississippi, 10.29.

"This striking dissimilarity in the regimen of streams of similar magnitude, and, with one exception, of similar climatic conditions, is entirely due to the reservoir action of the Great Lakes. Of that portion of the St. Lawrence drainage-basin which lies above Niagara Falls, viz., 265,095 square miles, 87,400 square miles, or almost one-third, is made up of the water-surfaces of Lakes Superior, Michigan, Huron, and Erie. One foot upon this immense area represents 2,436,000,000,000 cubic feet—greater than the excess of the late Mississippi River flood at Cairo above the bankful stage.

"The mean annual fluctuation of Lake Superior, based upon twenty-five years' observation (1871–1895), is 0.93 foot; of lakes Michigan and Huron, 1 foot; of Lake Erie, 1.16 feet. This fluctuation represents an annual storage of 2,419,000,000,000 cubic feet of water, equivalent to about 153,000 cubic feet per second for a period of six months. The maximum annual fluctuation during the above period is just about twice the above mean, and of course represents twice as much water stored.

"In addition to the annual fluctuation, there is constantly going on a periodic change which often requires several years to complete the cycle. As an illustration of this characteristic of the Great Lakes take the period of eight years from 1872 to 1879, inclusive, during which the mean annual level of the four upper lakes rose for a period of four years and fell during the following three years. The rise in mean level was, for Lake Superior, 1.03 feet; for Lakes Michigan and Huron, 2.02 feet; and for Lake Erie, 1.97 feet. The total storage represented by this rise of mean level was 4,000,000,000,000 cubic feet. The fall in mean level following the rise was, for Lake Superior, 1.63 feet; for Lakes Michigan and Huron, 1.46 feet; and for Lake Erie, 1.17 feet—equivalent to 3,627,000,000,000 cubic feet. After this fall the mean level began to rise again.

"The foregoing figures convey some faint idea of the magnitude of the storage of the Great Lakes, and of the way in which it operates to preserve a balance not only between the wet and dry seasons of each year, but between those cycles of wet and dry years which are continually recurring. These reservoirs absorb the flood-waters of spring and pay them out in the following dry season, thus preventing floods on the one hand and low water on the other. And while these seasonal changes are going on the lakes respond to the varying conditions of longer periods, levying upon years of more than average precipitation in order to maintain a flow in the outlets during the years of deficiency which are certain to follow.

"The result of this storage action of the Great Lakes is to produce a river system radically different in its general characteristics from nearly all other streams. Such conditions as high and low water, as elsewhere understood, are here entirely unknown. Commerce pursues its way through these lakes and rivers without serious hindrance

except when ice closes the way; and the river and harbor engineer has little to do with low-water problems or protection against floods, but rather with the deepening of harbors and connecting channels for an ever-increasing size of vessels and volume of commerce.

"The vital function which the fluctuation of levels, both annual and cyclic, plays in the economy of the Great Lakes is doubtless not generally appreciated even by the engineering profession. Only recently distinguished engineers have boldly asserted that this fluctuation of levels is an evil which must not be suffered to continue, and they have proposed plans by which it may be corrected. Yet nothing is more certain than that any curtailment of these fluctuations, either annual or cyclic, can be accomplished only by a corresponding curtailment at certain seasons of the discharge of the lake outlets.

"Besides the Great Lakes of the St. Lawrence basin there are many other natural reservoirs in various parts of the world. In order to convey some idea of their geographical distribution, magnitude, and regulating influence upon stream-flow, the following list of the more prominent examples is presented:

LIST OF PROMINENT EXAMPLES OF NATURAL RESERVOIRS.

Name of Lake.	River System.	Elevation above Sea-level. Feet.	Area. Sq. Miles.	Percentage of Area to Entire Watershed.	Storage in Billion Cubic Feet represented by a Fluctuation of 1 Foot.	Remarks.
Superior	St. Lawrence	601.6	31,800	39.5	886.5	Authority: Report of United States Deep Waterways Commission, 1896.
Michigan	St. Lawrence	581.2	22,400	32.9	638.4	
Huron	St. Lawrence	581.2	23,200	30.8	646.8	
St. Clair	St. Lawrence	575.3	495	7.3	13.8	
Erie	St. Lawrence	572.8	10,000	34.5	278.8	
Ontario	St. Lawrence	246.3	7,450	22.6	207.7	
Baikal	Yenisei	1,360.0	12,430	6.0	346.5	Encyclopædia Britannica. Watershed scaled from map.
Victoria Nyanza	Nile	4,000.0	27,000	24.0	752.7	Encyclopædia Britannica. Watershed scaled from map. Accuracy only approximate, as data are of doubtful authenticity.
Albert Nyanza	Nile	2,300.0	2,000	12.5	55.8	
Tanganyika	Congo	2,700.0	12,650	12.0	352.7	
Nyassa	Zambesi	1,600.0	9,000	24.0	250.9	
Titicaca	Desaguadero	12,600.0	3,200	17.0	89.2	Encyclopædia Britannica. Watershed scaled from map.
Geneva	Rhone	1,218.0	223	8.0	6.2	
Constance	Rhine	1,306.0	208	4.0	5.8	
Neuchatel	Rhine	1,427.0	92	11.9	2.6	
Como	Po	670.0	64	4.0	1.8	
Maggiore	Po	646.0	83	3.3	2.3	
Garda	Po	320.0	135	19.4	3.8	
Yellowstone	Missouri	7,741.0	139	15.9	3.9	United States Government reports.

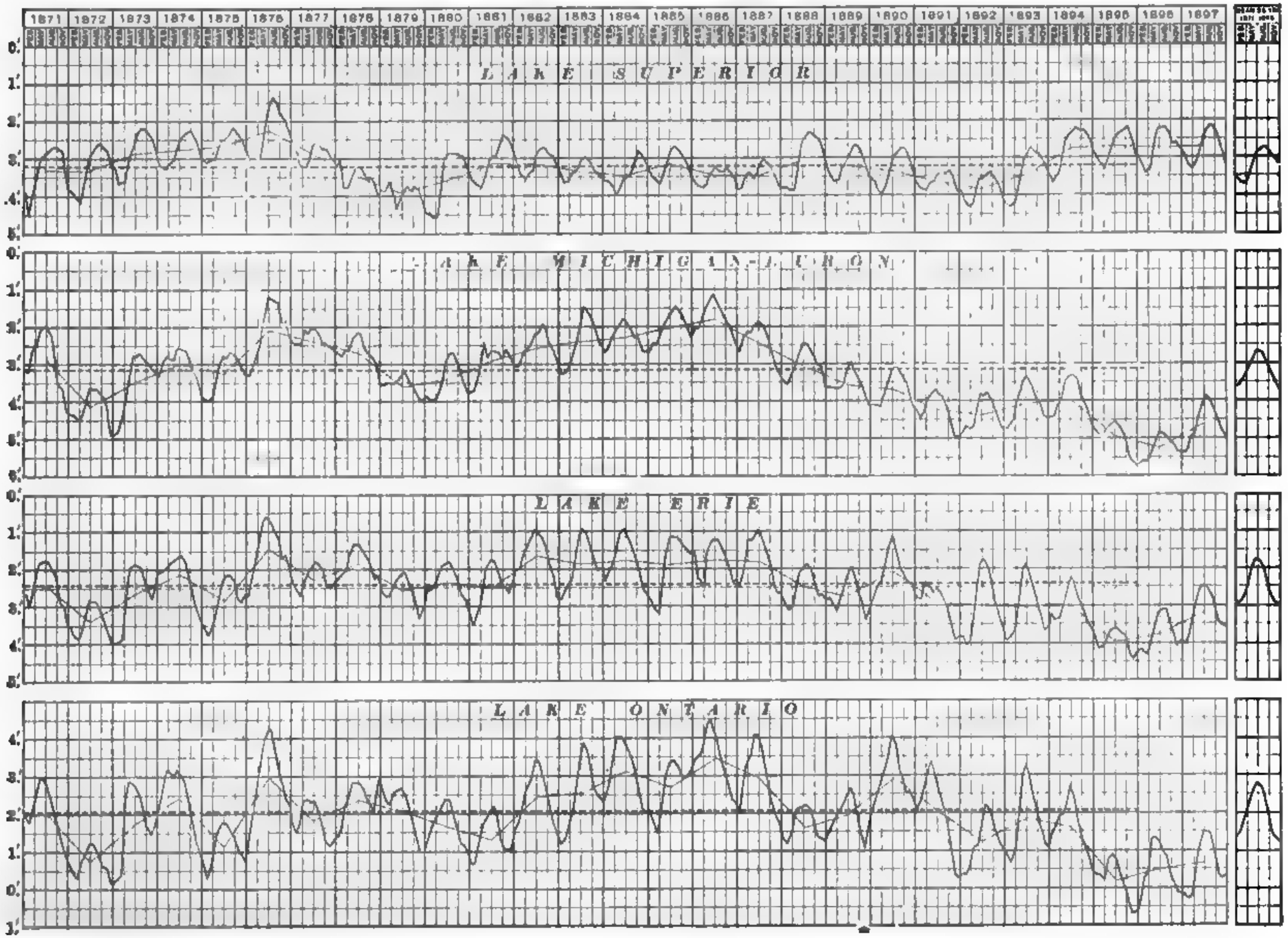
"The moderating influence of any of these lakes upon the streams below them is, of course, very great. Lake Geneva, for example, in the great flood of 1856 discharged only 11,400 cubic feet per second at the maximum, as against 56,480 cubic feet which it was receiving from its watershed.

"In Italy the lakes on several of the northern tributaries of the Po have long been noted for the control which they exercise over the streams flowing through them. The

CHART SHOWING THE OSCILLATIONS OF LEVEL AND OF MEAN ANNUAL LEVEL OF THE

GREAT LAKES OF THE ST. LAWRENCE BASIN

FROM 1871 TO 1897 INCLUSIVE, WITH MEANS OF TWENTY-FIVE YEARS FROM 87 TO 1895 INCLUSIVE.



(To face p. 104.)

violent and destructive floods which are characteristic of other tributaries of the Po are largely absent from those streams which flow through the lakes.

"The flow of the Rhine in its upper source is said to be subject to much less variation than other streams similarly conditioned except as to natural reservoirs.

"There are many thousands of other lakes scattered over the globe that act as regulators of the streams which drain them, their efficiency in this respect being proportional to the percentage which their areas bear to the tributary watersheds. Certain it is that the aggregate influence of these reservoirs is very great, and the striking difference often noted in the characteristics of the flow of streams with similar watersheds may largely be traced to this cause.

Artificial Reservoirs.—"While it is impracticable to imitate nature on the scale of her own work in the construction of reservoirs, her example has nevertheless been followed very extensively on a smaller scale. In fact, works of this character have been built for a variety of purposes since the remotest antiquity. The storage of water for feeding canals is a prominent example. The greatest reservoir systems yet constructed have been designed to maintain the navigable condition of natural waterways. Many reservoirs have had as a prominent reason for their construction the prevention of floods in the valleys below them, although this has seldom if ever been an exclusive reason. Storage of water for city supply, the development of power, and other industrial uses, is one of the most familiar of modern enterprises. Finally the field of irrigation, which already presents many examples of great reservoirs, bids fair to outstrip all other fields in the production of works of this character. In all these examples of reservoir construction the purpose has been to correct the inequalities of nature—to prevent the rapid and destructive flow of rivers at seasons when not needed, and to augment and re-enforce that flow when the need does exist.

"One of the most extensive artificial systems ever built is to be found in Russia at the head waters of the Volga and Msta rivers. The Volga River, the greatest in Europe, 2325 miles long, and navigable nearly its whole length, rises in the province of Tver, within 200 miles of St. Petersburg, and empties into the Caspian Sea in the opposite extremity of European Russia. The Msta River has its sources interlaced with those of the Volga, but flows in the opposite direction, and its waters find their way, through the Volkhoff River, to Lake Ladoga, and ultimately to the Baltic Sea.

"The sources of the Volga and Msta are in a flat, marshy, wooded country, about 665 feet above sea-level, covered with innumerable lakes, presenting conditions not unlike those which prevail at the sources of the Mississippi River in our own country. For a long period in the past these two river systems were connected by artificial waterways, and the seaport of the upper Volga was upon the Baltic. The extreme low water which is characteristic of the Volga and other Russian streams prevents navigation in their natural condition except in seasons of high water. To ameliorate this condition, advantage was early taken of the exceptional reservoir facilities offered by the lakes referred to, and dams of a cheap character were constructed across their

outlets. The reservoir system has now been developed to great perfection and effects an important improvement both in the Volga and the Msta, rendering them navigable for nearly three months longer than they would be without this aid.

"These reservoirs store about 35,000,000,000 cubic feet of water in all, of which 20,000,000,000 can be used in the Volga and 20,000,000,000 can be turned in the other direction, there being apparently a storage of about five or six billions that can be used in either direction. The largest and most important of these reservoirs, and one of the largest in the world in point of capacity, although insignificant in depth and containing-dam, is the Verkhnevoljsky reservoir. So slight is the fall of the stream in this region that, although the dam produces a maximum elevation of water-surface at its site of only about 17.5 feet, the water backs up a distance of about 60 miles and includes several lakes. The low-water season capacity of this reservoir is about 14,000,000,000 cubic feet, and the average season storage is much greater. Its effect upon the low-water flow of the river below the dam is to raise its normal surface 2.8 feet at Rjef, 96 miles below; 1.4 feet at Tver, the mouth of the Tvertsa, 212 miles below; and 0.14 feet at 410 miles below. At the mouth of the Tvertsa the storage of the Zavodsky reservoir comes in and helps out the navigation below. The total navigable distance on the Volga over which the beneficial influence of these reservoirs is felt is upward of 450 miles.

"On the Msta slope there are no fewer than ten important reservoirs, all of them being on the sites of natural lakes, the total storage aggregating about 14,000,000,000 cubic feet. As already stated, about 6,000,000,000 cubic feet of storage which really lies on the Volga slope, including the Zavodsky reservoir, formerly was and still can be turned into the Baltic drainage. This entire system of summit reservoirs that can be used to feed the Msta is called the Vychnevolotsky system. It affords material improvement to the navigable condition of Msta and Volkhoff rivers during the period of low water.

"The system of reservoirs just described is certainly a great success, and upon it much of the prosperity of the surrounding country depends. It is probably the most complete example in the world of the joint results of flood prevention and the improvement of navigation produced by artificial reservoirs. It has an importance, however, which it could not have in this country, even with equal physical advantages, for railroads here do a far greater proportion of the transportation business than in Russia. But the example shows how far favorable natural conditions can be made to improve the low-water conditions of streams.

"The largest artificial-reservoir system ever yet constructed is that at the head waters of the Mississippi River. The natural conditions prevailing in that region are very similar to those in Russia just described. The country about the sources of the Mississippi, where the reservoirs are constructed, is about 1200 feet above sea-level. It is dotted with an immense number of lakes, the total number having been estimated as high as a thousand. Some of the larger of these lakes afford exceptionally favorable

opportunities for the inexpensive storage of water. The dams required are low structures, but the area over which the water is raised by them is so extensive that the cost per unit of volume stored is probably the smallest ever yet realized.

"These remarkably favorable natural conditions for the storage of water have long attracted public attention and were made the subject of an able official report by Gen. G. K. Warren as early as 1870. Exhaustive surveys followed at a later date, and in 1881 actual construction was begun. Up to the present date there have been constructed five reservoirs, each with an aggregate capacity of 93,400,000,000 cubic feet, at a total cost of $678,300.

"The average annual storage of these reservoirs is estimated at about 40,000,000,000 cubic feet, equivalent to about 5200 cubic feet per second for a period of ninety days. This supply is estimated to increase the gauge height at low water at St. Paul, 357 miles below, from 1 to 2 feet.

"The original investigations, embracing the States of Minnesota and Wisconsin, indicated a practicable storage in Minnesota of 95,000,000,000 cubic feet, and in Wisconsin of 79,000,000,000 cubic feet, or a total in the two States of 174,000,000,000 cubic feet. There is probably little doubt that the system could be extended so as to secure a storage of 150,000,000,000 cubic feet in the two States, an equivalent of about 20,000 cubic feet per second for ninety days. From the results already obtained, it is probable that this storage would not cost above $2 per acre-foot. The effect upon the navigable stage of the river would, of course, vary with the locality considered, and would diminish rapidly with the distance down stream. But considering that such an improvement is of the most permanent character, depending only upon the maintenance of the dams for its perpetuity, the above cost cannot be considered excessive when compared with the vast outlay for the mere temporary improvement of these rivers by present methods. A permanent increment of from 10,000 to 20,000 cubic feet per second to the low-water stage of even so large a stream as the Mississippi River is not to be passed over as a matter of small importance.

"The Volga and the Mississippi rivers constitute the only two *systems* of artificial reservoirs yet constructed, and the only ones designed to improve the navigable condition of streams in their natural condition.

"The construction of reservoirs to feed artificial waterways has been resorted to extensively, particularly in France, and to a considerable extent in this country. Inasmuch as the expenditure of water in canals is a matter of very exact determination, the storage required for this purpose can generally be estimated with great definiteness.

"The construction of reservoirs for municipal purposes is too common a matter to require particular mention. It is sufficient to say that nearly every city in the world of above 100,000 population has storage facilities of greater or less extent to help out its water-supply.

"The principal development of storage reservoirs for irrigation purposes has taken place in Spain, in France and Algiers, in India, and in the United States.

"For such industrial purposes as the operation of factories and the like many reservoirs have been constructed both in France and in this country. They are generally of small capacity, and costly per unit of water stored, but profitable on account of the great use made of the water. Some of these reservoirs serve an important purpose in protecting the valleys below from floods.

Effects on Floods.—"Every reservoir built along the course of a stream is, to some degree, a protection against floods in the valley below. The extent of this protection depends, of course, almost entirely on the ratio of its capacity to the flood discharge. A reservoir that can store the entire flow of a stream is an absolute protection against floods for a considerable distance below. It is difficult to propose any general rule for the extent of this control, but, assuming a general similarity of watershed, it would seem not unreasonable to say that it ought to be decisive to at least such a distance below as will give an additional watershed to a stream equal to twice that above the reservoir. This is simply saying that, in the general case, the reduction of a flood wave by one-third of its volume will rob it of its destructive character.

"But in a great many cases this control extends very much farther. For example, in the case of a flood caused by the rapid melting of snows in the mountains, reservoirs below which can impound this flood will protect the entire valley so far as its destructive influence would otherwise have reached. When it is remembered that the volume of a destructive flood is only a part—probably always less than half—of the total flow of a year, it will be admitted that a storage capacity equal to one-fourth of the run-off, well distributed throughout a watershed, will practically eliminate the evil effects of floods in its streams, and supply a percentage sufficient for the purposes of irrigation.

"It is not necessary, though important, that a reservoir should be empty when a flood comes. Even if full, it still moderates the flow of the stream below, the effect varying directly with the superficial area of the reservoir when full, and inversely with the capacity of the spillway. In this respect it acts precisely as does a natural lake. For example, if the spillway of a reservoir or the outlet of a natural lake be of such dimensions as to require a considerable increase in the depth of water to give much of an increase of discharge, every increment of this depth of outlet means also an increment of the same depth over the entire reservoir. A flood passing such a reservoir will be reduced by the storage resulting from this increment, and before it can produce a full discharge it must fill the reservoir to the necessary height above the bottom of the spillway. A large reservoir is, therefore, even when full, always a perfect protection against sudden floods. In the case of long-continued floods it greatly retards the arrival of maximum effect and gives ample notice of its approach.

"In fact, this is a very important feature of reservoir action, even where the capacity of the reservoir is not sufficient entirely to prevent the flood. It does prevent freshets—that is, sudden floods—and in smaller streams it is often the suddenness quite as much as the magnitude of floods that causes damage and loss of life.

"A reservoir ceases to be any protection if a flood continues long enough to fill it

to such a height that the discharge at the outlet is equal to the entire inflow. The same is true of the restraining influence of forests. A sudden and heavy precipitation of short duration, which might produce a severe freshet in a deforested region, would probably experience considerable retardation, and even reduction, if it should fall upon a forest-covered region; but if the rains continue long enough to exhaust the retentive capacity of the forest soil, to fill all the springs and replenish the ground storage, then forests cease to be any protection whatever. In fact, the presence or absence of forests in a vast watershed like that of the Mississippi River is without appreciable influence upon the great floods.

"In the case of floods, which are the results of combinations of discharges from the various tributaries, reservoirs may actually operate to increase the combination. Take for example the natural reservoirs at the sources of the Mississippi. While they restrain the flood excess in that stream, they keep up a heavy flow for some time after the flood has passed. If this larger flow happens to come in with a flood crest at the junction of some tributary below, it will actually increase the combination over what would have been the case without the reservoirs. In the French investigations, presently to be described, the dams proposed for restraining floods were to have open sluiceways without means of closing them. In the ordinary flow of the stream all the water could pass through. But they were to be so proportioned that when the flow should pass a certain point the surplus would be retained in the reservoir, the outflow being always limited by the capacity of the open sluices. The arrangement was, therefore, precisely like that of a natural lake without a dam across the outlet. The outflow could never be entirely restrained, and it would increase in proportion to the height of water in the reservoir. Now, in the case of a large stream like the Rhone, where flood *combination* is the really dangerous thing, it was found that these reservoirs, had they actually been constructed, would have increased certain floods. They would have maintained a heavy retarded flow on some tributaries which in their natural condition would have entirely run out before the arrival of floods from other tributaries. As it happened, this retarded flow in the one case would have come upon a flood crest in the other, and would actually have increased the natural combination. This, of course, could not be true of reservoirs with closed sluices, unless, as above stated, the reservoirs were entirely filled with the flood passing over them.

"It is, therefore, clear that the efficiency of reservoirs in moderating great floods would have to be a matter of judicious management in controlling combinations quite as much as of actual capacity.

"Another matter to be noted in this connection is that flood protection and industrial use are not entirely compatible objects. To serve the former purpose alone the reservoir should be kept empty until the flood arrives, so that its whole storage may be available. But this might leave the reservoir only partly filled when its supply is needed for other purposes. Generally, therefore, the whole capacity of reservoirs built

for these joint purposes cannot be counted on for flood protection. It would probably be unsafe to allow a higher efficiency in this respect than 50 per cent.

"For reasons to be fully considered further on, very few, if any, reservoirs have been built for the exclusive purpose of protecting against floods the valleys below them; but there are numerous examples where this has been an important consideration in their construction. Two cases may be cited in France. The celebrated dam at the Gouffre d'Enfer, on the river Furens, near St. Etienne, was built largely to protect St. Etienne from the destructive freshets of the Furens. It was of course expected to make use of the stored water for industrial purposes, which in a thickly populated district could not but be important. As to the results obtained, the expectations in regard to flood protection have been fully realized.

"The Ternay Dam likewise had as an important motive for its construction the protection of the town of Annonay from the floods of the Ternay, although in this case, as in that just cited, industrial uses of the stored water were considerations of great weight. The result of this work, as to flood protection, has been a success.

"There are certain reservoirs in Germany, as that at Dahlhausen, on the Wappen, and another in the valley of the Bever, which serve very much the same purpose as do those at Furens and Ternay in France, and exercise an important influence upon the floods in their respective valleys. Various similar works have been constructed in other parts of Europe, but all have other motives in addition to that of flood protection to justify their construction.

"The systematic creation of a comprehensive system of reservoirs on any river for the sole purpose of mitigating the severity of floods has never been undertaken. The subject has, however, received exhaustive study, and some examples of such studies will therefore be of importance in this connection. By far the most important of these studies, as might have been expected, is to be found in France. It took place during the reign of Emperor Napoleon III., as a result of the floods of 1856. These floods were among the greatest and most destructive that had ever visited France, and aroused a great deal of interest in the question of their future prevention. Among the various proposals which were brought forward at the time was that of constructing reservoirs at the head waters or on the tributaries of the various streams, among which particular attention was given to the Rhone, Garonne, and Loire. These investigations were ordered by the Emperor under date of July 19, 1856, and resulted in the most exhaustive analysis of the whole subject and in reports of great scientific value. They embraced the three streams above mentioned, and the result was adverse to the project so far as the Rhone and Garonne were concerned and favorable as to the Loire. A brief résumé of the reports will here be given.

"*Rhone River.*—The damages wrought by the flood of 1856 in the Rhone Valley were extraordinary. Over 540,000 acres of rich valley lands were submerged and the newly started crops were destroyed. The injury to bridges, dikes, revetments, and other river works was very great, as was also the destruction to the towns and cities

situated along the stream. The total damages on French soil in the Rhone valley were estimated at not less than $6,000,000.

"So great a disaster in one of the most populous sections of France naturally led to inquiries into the possibility of preventing a recurrence of it. Napoleon III., who had taken a great interest in public works and favored a liberal extension of them, ordered an elaborate investigation of the subject; first, as to the immediate protection of great centers of population, and second, as to the practicability of 'modifying the régime of great watercourses for the protection of the bottom lands by a diminution of floods by means of reservoirs established near the head waters of the tributary streams.'

"The first part of the programme, viz., the protection of the river towns by works intended to confine the floods to proper limits, was reported practicable at a total cost of about $4,000,000. The second part of the programme, viz., the question of reservoir construction, was considered in great detail and with a thoroughness of study which makes it the best existing example of what may be expected from similar works in other localities.

"The river Rhone has a total length of about 447 miles and a watershed of about 36,670 square miles. Three hundred and thirty-six miles above its mouth is Lake Geneva, an immense natural reservoir, with an area of 223 square miles. Below Lake Geneva, at the distances given, the main stream receives the following important tributaries:

"The Arve, 1¼ miles below the outlet of the lake, drainage area 2422 square miles; the Ain, 110 miles, drainage area 1355 square miles; the Saône, 131 miles, drainage area 11,019 square miles; the Isère, 179 miles, drainage area 4360 square miles; the Ardèche, 225 miles, drainage area 938 square miles; the Durance, 272 miles, drainage area 5716 square miles. The drainage area of all the other tributaries is about 7200 square miles. The drainage area tributary to Lake Geneva is 2663 square miles, of which 2078 square miles pertains to the Rhone above the lake.

"The flood of 1856 in the valley of the Rhone was practically a simultaneous affair in all parts of the valley. Only in the upper portions was there any apparent progression. The maximum occurred at the mouth of the Arve thirty-six hours before it reached the mouth of the Ain, 108 miles below; but for the entire remainder of the river the maximum occurred on the same day, with a variation of only a few hours. The causes that led to the flood were therefore operating throughout the entire valley, swelling all the tributaries at once, and in consequence causing a simultaneous elevation of all portions of the main stream.

"The following table shows some of the characteristics of this flood, and gives an admirable illustration of the effect of natural reservoirs in moderating the flow of a stream. It will be observed that the flow of the Rhone just above the Arve indicates a run-off of only 4.3 cubic feet per second per square mile. As a matter of fact, the upper course of the Rhone was discharging into the lake 42,360

Name of Stream.	Drainage Area. Sq. Miles.	Discharge. Second-feet.	Rate of Run-off per Square Mile per Second.
Rhone above the Arve	2,663	11,472	4.3
The Arve	751	24,710	31.0
The Ain and smaller tributaries below Arve	3,777	161,674	43.0
Saône and smaller tributaries below Ain	11,264	49,420	4.0
Isère and smaller streams below Saône	6,079	92,662	15.0
Ardèche and smaller streams below Isère	2,916	80,307	27.0
Durance and smaller streams below Ardèche	7,232	70,600	10.0
Durance to the sea	1,569		
Entire river	36,352	490,670	14.0

cubic feet per second, or 21 cubic feet per second per square mile, which would indicate for the entire watershed above the Arve, including that of Lake Geneva itself, 56,480 cubic feet per second. The storage of Lake Geneva accounts for the difference, and actually reduces the flow of the upper Rhone by about 45,000 (56,480−11,472) cubic feet per second.

"Again, it will be seen that the discharge of the great tributary, the Saône, is at a rate of only 4 second-feet per square mile. Although there is no lake forming a reservoir in this valley, as in that just described, the slope of the lower portion of the valley for 100 miles above Lyons is so slight that floods do not pass off rapidly, but fill up the bottoms over 166 square miles to a depth of 10 feet or more, giving a storage of upward of 50,000,000,000 cubic feet. If the flow of this stream had been as great per square mile of watershed as that of the Rhone above Lyons, without the moderating effect of Lake Geneva, it would have been about 363,000 cubic feet per second instead of its actual flow of about 50,000 cubic feet. Without the storage effects of Lake Geneva and of the Saône valley, the discharge of the Rhone at Lyons would have been about 600,000 cubic feet instead of its actual discharge of less than 250,000 cubic feet. The great influence of these two natural reservoirs in moderating the flood discharge of the Rhone at Lyons is thus clearly apparent, and it is evident that without them the range between high and low water, or the ratio of minimum to maximum discharge, would be much greater than it actually is. It would not, however, be correct to infer from this that the destructive power of the great floods of the Rhone would, under the above supposition, increase in the same proportion as the discharge itself. Nature adapts the channels of streams to the work required of them, and if the flood flow of this river were greatly increased undoubtedly it would carve out a deeper and wider bed, and would carry away, within the limits of safety, a much larger volume of water than it does at present. Thus, while the absence of these natural reservoirs would, probably, to some extent increase the destructive power of the floods of the Rhone, it would not do so in anything like the same proportion in which it would augment the flood discharge at Lyons.

"When, in the course of their investigations, the French engineers undertook to supplement the effect of these natural reservoirs by artificial ones, they were confronted

with practically insuperable obstacles. Nature had not provided suitable localities, and an exhaustive study of the whole basin gave only the following meager results:

"Lake Geneva could be so dammed at the outlet as entirely to cut off its discharge at the time of flood.

"The Arve and its tributaries, being mostly torrential streams, afford very few good reservoir sites. In fact only one was deemed worthy of consideration, and its capacity was only 706,000,000 cubic feet. This would be of no use to the upper Rhone, which flowed between high banks not subject to overflow, and by the time it reached Lyons its effect would be wholly inappreciable. The reservoir would cost $400,000, besides the destruction of valuable bottom lands. This project was therefore not considered practicable.

"The next site in passing down stream is what is known as the Lac du Bourget, situated to the east of the river and forming a kind of natural reservoir in times of flood. It was proposed to carry this natural action still farther by damming the Rhone. Its natural storage capacity is 3,350,000,000 cubic feet, and this could be increased to 5,824,000,000 cubic feet. It was calculated that this storage would diminish the flow of the Rhone at the moment of flood by 35,000 cubic feet per second, and would diminish the height of the flood at Lyons by 2.3 feet. The cost of this work would be about $4,000,000.

"No further reservoir sites of importance were found above the junction of the Ain. In this valley there are several feasible sites, whose aggregate capacity would be nearly 2,000,000,000 cubic feet. The cost would be about $1,400,000. The estimated effect at Lyons on a flood like that of 1856 would be to reduce the height of the flood by about 1 foot.

"No reservoirs were recommended for the Saône, because none that could be found would have any appreciable effect as compared with the vast natural reservoir formed by the lower part of the valley already alluded to, and would have almost no influence on the discharge of the main stream at Lyons.

"Below Lyons the immediate valley of the main stream offers no opportunities for large reservoirs.

"The first large tributary on this section of the river, the Isère, was carefully studied, but no situations were found which were considered favorable. The alluvial and unsatisfactory nature of the foundation for dams, the necessity of condemning valuable bottom lands, the small aggregate result possible of attainment under the most favorable circumstances, rendered the project wholly unadvisable.

"The valley of the Ardèche likewise contains no feasible reservoir sites.

"On none of the other tributaries were suitable sites found until the Durance was reached. The valley of this stream, which is one of the largest affluents of the Rhone, offers several good sites, and it was found practicable to store 11,366,600,000 cubic feet of water at a cost of about $6,600,000. The result, however, was altogether insignificant. The Durance enters the Rhone far down the valley of that stream, where its

flood discharge is already very great. The effect of the proposed reservoirs on the flood of the Rhone immediately below the junction would be to diminish its height by less than 1.3 feet.

"The following tabular summary shows the magnitude and cost of the foregoing works:

Reservoir.	Capacity. Cubic Feet.	Cost.
Lake Geneva	2,294,500,000	$1,000,000
Valley of Arve	706,000,000	400,000
Lake du Bourget	5,824,000,000	4,000,000
Valley of Ain	2,000,000,000	1,400,000
Valley of Durance	11,366,600,000	6,600,000
Total	22,191,100,000	$13,400,000

"The result of these works and of this expenditure may be summarized as follows:

"Over the 24,700 acres of submergible lands the depth of overflow would be reduced from 2.2 to 3.2 feet. But this would not entirely prevent submersion, and the necessity for dikes would exist as before. Through Lyons the flood height would be reduced possibly 3 feet, but would save none of the special works of protection and would but slightly diminish their cost. From Lyons down the diminution of height of flood would be as follows: Below mouth of Saône, 1.3 feet; at Tournon, 0.8 foot; at Valence, 0.6 foot; below Valence, inappreciable. The effect of the proposed reservoirs in the valley of the Durance on the floods of the Rhone below the junction of the two streams would be to diminish the flood height at Beaucaire 1.3 feet; at Arles, about 0.5 foot; below Arles, not at all.

"The effect of these reservoirs, therefore, although considerable in absolute magnitude, would not be sufficient, in comparison with their great cost, to justify adoption and the project was reported upon adversely by the engineers.

"This report does not deal with the low-water flow of the Rhone at all, nor with the effect which this storage would have upon the interests of navigation. Undoubtedly it would be much greater than in the control of floods. For example, the 10,000,000,000 cubic feet of water that could be stored upon the upper Rhone and the Ain would provide a flow of about 4000 second-feet for one month, or 1300 second-feet for three months, and could undoubtedly be so regulated as to be of considerable advantage to navigation. The increase for a period of one month only over the low-water flow at Lyons would be nearly 50 per cent.

"*Garonne River.*—Similar studies to those just described were also made in the case of the Garonne, which had likewise suffered severely from the floods of 1855 and 1856. Without reviewing these studies in detail, the following conclusions may be stated in the language of the report:

"'Reservoirs, when their capacity is great enough, have a very powerful effect in diminishing the flood discharge of the streams on which they are built, but their influence diminishes enormously with distance; and inasmuch as suitable sites can be found only

in the mountainous regions, far removed from the bottom lands to be protected, it may readily be seen how slight must be their influence on the flood heights in the valleys far below. . . . To reduce such a flood [as that of 1855] to the height required in order to contain it within the proposed system of dikes would require a storage capacity exceeding 33,000,000,000 cubic feet, and would cost $24,000,000. . . .

"'Other objections of a fundamental character as to all reservoirs have already been stated.

"'The conclusion arrived at, therefore, is that the idea of reducing the floods of the Garonne by means of artificial reservoirs must be abandoned.'

"*Loire River.*—The studies devoted to this question in the case of the river Loire were more favorable to the use of reservoirs. This was owing to the more favorable conditions which prevail on that stream. The main stream is formed by the union of the upper Loire and the Allier near the city of Nevers at the Bec d'Allier. The Loire is subject to the most extreme variations in the matter of flow. At the junction of the two streams, for instance, it varies from about 10,000 cubic feet per second to 350,000 cubic feet. The floods in the lower river are ordinarily rendered harmless by the arrival, at different times, of the floods from the various affluents; but when the conditions cause the simultaneous arrival of flood-crests from several tributaries the results are liable to be of the most serious character.

"The floods of the Loire River have always been a matter of great moment to the interests of the valley, and have led to extensive works for their control. In the studies above referred to the use of reservoirs on certain portions of the streams was recommended, viz., upon the upper Loire and the Allier. These two streams, heading in the south-central part of France, flow north nearly parallel to each other at distances scarcely ever 50 miles apart. Their drainage areas are 7000 square miles and 4500 square miles, respectively. The geographical, geological, and meteorological conditions are essentially the same for the two streams. They rise in high land some 4500 feet above the level of the sea. The mountain slopes are steep and the soil of a very impervious character. The result is that the run-off responds quickly to the rainfall; floods are quick and of short duration, and the curve of the flood-wave at any point is sharp in character, i.e., very high compared with its length. The conditions in the two valleys are so similar that the crests of floods reach the junction very nearly at the same time, being only two or three hours apart in the great flood of 1856. The curves of discharge of the two streams, both accentuated in character, are superimposed upon each other, producing a curve of relatively the same relief, but absolutely nearly twice as pronounced, as in the case of either tributary.

"The union of two such considerable tributaries with floods of the nature above described gives character to the flood-wave of the united stream for a great distance below, or until the accession of tributaries reaches an extent that may exert a marked modifying influence. But it is stated that the sharp form of the wave does not entirely disappear even to the mouth of the river.

"The flood conditions, therefore, prevailing from Nevers for a long distance down are those of extreme height but short duration. Were it possible to cut off the upper part of this curve and retain the water which it represents, thus reducing the flood curve to the normal form of the other principal tributaries, the floods would be brought within limits which would keep them between the dikes proposed to be constructed along the river.

"An examination of the valleys of the upper Loire and the Allier disclosed the following possibilities as to the storage of water:

"In the valley of the upper Loire twenty-two reservoirs would store about 8,250,000,000 cubic feet of water, and would reduce to 111,653 cubic feet per second the flood-flow, which, without these reservoirs, would be 153,555 cubic feet per second at Bec d'Allier. In the valley of the Allier sixty-three reservoirs, storing about 10,000,000,000 cubic feet, would reduce to 104,664 cubic feet per second the flood-flow which, without the reservoirs, would be 167,675 cubic feet per second. The total reduction would therefore be about 95,000 cubic feet per second from a total flood-flow of 320,000 cubic feet per second, or a reduction of about 30 per cent. This would deprive floods of their destructive character as far down as to the mouth of the Cher, a distance of about 180 miles below the junction of the two streams.

"It is thus seen that the peculiarly favorable conditions existing on the upper Loire make possible an important reduction of flood-height for a certain length of the river below Nevers. In the upper valleys, near the reservoirs, their effect would, of course, be far greater, and would effectually remove the possibility of flood.

"These proposed works, however, were of great magnitude, estimated to cost over $13,000,000, and they have never been carried out.

"A very interesting and exhaustive investigation, similar to those just described, has been conducted by German authorities in the valley of the river Alb. The study goes into too much detail to be given here, but its general conclusions are so in line with those of the French engineers that they cannot fail to be of interest. The report says:

"'It cannot be denied that for the head waters of rivers, and also for the territory of small streams, the question might be solved. The Government of Würtemberg investigated the matter and found that high floods could be prevented by means of reservoirs, but that the benefit would not be commensurate with the cost. . . . This investigation [the prevention of floods on the Alb] has proved that the construction of reservoirs for the purpose of keeping back the high water of the Alb, although possible, and with no doubt of their effectiveness, is still unjustifiable on account of the enormous cost.'

"And again:

"'There seems to be no doubt that the construction of a system of reservoirs on a large scale in the valley of the Alb is inexpedient, on account of the great cost.'

"Particular emphasis is placed upon these studies, because they disclose the true obstacle to the use of reservoirs for the sole purpose of flood prevention. It is the *cost*,

not the physical difficulties, which stands in the way. It may be stated that as a general rule a sufficient amount of storage can be artificially created in the valley of any stream to rob its floods of their destructive character; but it is equally true that the benefits to be gained will not ordinarily justify the cost.

"The reason for this is plain. Floods are only *occasional* calamities at worst. Probably on the majority of streams destructive floods do not ·cur, on the average, oftener than once in five years. Every reservoir built for the purpose of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. One acre permanently inundated to rescue three or four acres from inundation of a few weeks once in three or four years, and this at a great cost, could not be considered a wise proceeding, no matter how practicable it might be from engineering considerations alone. The cost, coupled with the loss of so much land to industrial uses, would be far greater than that of levees or other methods of flood protection.

"In fact, the examples of natural reservoirs already cited, while they show conclusively the vast beneficial influence of large reservoirs upon the flow of streams, also disclose the fatal obstacle to their successful imitation by man. In only very few places has nature prepared sites where man can erect works which will create large bodies of water, and even if she had done so the gain from utilizing them would not equal the loss. The reservoir system of the Great Lakes involves the perpetual withdrawal from agriculture and industrial uses of an area nearly twice the size of the State of New York. Were these areas not covered with water, but occupied as the surrounding country now is, yet so fitted by nature that man, at slight expense, could convert them into great lakes, as at present, the utter impossibility of such a measure is evident at a glance. And so it will be found in general that the surface of the earth, where reservoirs could be built on an extensive scale, is liable to be of more value in its present condition than it ever could be if covered with water.

"The construction of reservoirs for flood protection is not, therefore, to be expected, except where the reservoir is to serve some other purpose as well, and inasmuch as such purposes are not ordinarily extensive enough to develop *systems* of reservoirs, upon which, rather than upon isolated works, the control of great floods depends, this large control is hardly one of the possibilities of the future. The only probable exception is that of a reservoir system on the watershed of the Missouri River, treated of in the next section of this report.

"For flood protection in isolated cases, however, and on a relatively small scale, reservoirs will undoubtedly continue to be built, particularly when they serve other purposes as well. From this point of view they will always be projects of public importance. The idea is well presented by the distinguished French engineer, P. Guillemain, former inspector-general of public works in France, who holds that the creation of reservoirs is of public utility in nearly all cases, either in flood prevention or in re-enforcing low-water flow, and that whenever special interests, such as industrial uses, irriga-

tion, and the like, exist that will justify their construction, they become legitimate, subjects for Government adoption.

"**The Floods of the Mississippi and the Missouri.**—A belief that it is within the range of possibility to diminish materially the great floods of these rivers by means of reservoirs upon its tributaries has long been held. In a work well known in its day (The Mississippi and Ohio Rivers, by Charles Ellet, Jr., published in 1853), the author advocates this view with great vigor, and had his data been as correct as his argument he would have made out a good case. The subject was briefly reviewed by Humphreys and Abbot in their report upon the Mississippi River (1861), and the views of foreign engineers upon this method of river regulation were cited at considerable length. Although the authors of this report pronounced the scheme impracticable so far as the Mississippi is concerned, the idea has, nevertheless, continued to have its advocates from that day to this. It has occasionally found expression in public documents or acts of Congress. In the voluminous report of the Senate Committee on Irrigation, which forms Senate Report No. 928, Fifty-first Congress, first session, the committee say:

"'It is confidently believed that, with restraining dams to hold back the water of the numerous lakes found at the head waters of the various tributaries of these rivers, and reservoirs constructed at other suitable points, together with the aid of the natural flow of the streams, a very large extent of country, now comparatively worthless, could be made exceedingly productive, while the floods in the lower Mississippi would be greatly alleviated.'

"During the past year two investigations have been ordered by Congress, having as one of their objects an examination of this reservoir question. Among engineers there are not a few of reputable standing in their profession who hold similar views to those expressed in the Senate report quoted above. With the general public the idea is almost an axiom, and it finds constant expression in the press, particularly when a great occasion, like that of the recent Mississippi floods, calls attention to it. It has therefore seemed important to devote some especial care to the subject, and very soon after taking up the study I arranged to have Mr. James A. Seddon, United States assistant engineer, compile existing data on the Mississippi floods in such form as to present the subject in its entire magnitude so that it can be readily understood.

"Few people have any adequate conception of either the origin or the magnitude of great floods like those on the lower Mississippi. It is a common error to think that they come largely from the melting snows in the mountains. Yet the floods of the Mississippi nearly all come at seasons when the flow from the mountains is very small. In the greatest known flood of the Mississippi at St. Louis, that of 1844, a large part of which came from the Missouri, the latter stream was found by pilots to be in low-water stage above Sioux City. On the occasion of the late heavy flood in the Mississippi, when at its maximum stage, the Arkansas carried practically no water across the Kansas-Colorado line, the Platte did not run above 2000 cubic feet per second at North Platte, Neb., and the upper Missouri and Yellowstone were both in low-water stage.

The floods of the Mississippi do not come from this direction. They are formed by the heavy rains in the low regions east of the ninety-eighth meridian, and very largely come from east of the Mississippi itself. The great controlling element, in fact, in all the lower river floods is the Ohio River.

"The magnitude of these floods also depends very largely upon fortuitous combinations of the floods in its tributaries. No single flood from any one of these tributaries, except the Ohio, can produce serious consequences in the main river. But if two or more of them discharge excessive floods in the main stream simultaneously, then it is that great disasters follow. Very fortunately, nature has caused these flood-waves to arrive generally at different periods, and the more disastrous combinations are not of frequent occurrence.

"It is apparent, therefore, that a reservoir system which should exercise any appreciable influence on the lower-river floods must embrace the three great upper tributaries, and particularly the Ohio. What the magnitude of the storage required would have to be may be inferred from the fact that the total discharge of the Mississippi at Cairo, above the bankful stage, during the late flood, was 2,368,000,000,000 cubic feet, or 4250 square miles 20 feet deep, the assumed average depth of reservoirs. The largest artificial reservoir ever built—viz., that at Lake Winnebigoshish, Minn.—has a capacity of 45,000,000,000 cubic feet. To store all this excess would take fifty-two such reservoirs.

"While it might seem at first thought that this amount of storage could be found, still it would be very difficult to find it. Particularly on the upper Ohio and its southern tributaries favorable sites are understood to be of rare occurrence. It is probable, however, that in all the watershed of the Mississippi sites could be found that would insure a reduction of a flood discharge at Cairo like that of 1897 by one-fifth of its maximum. The ease with which the writer was able to find storage amounting to 11,000,000,000 cubic feet in the State of Ohio at the very head waters of streams along the divide between Lake Erie and the Ohio convinced him that the natural facilities for storage are rather greater than is commonly supposed.*

"As already stated, the difficulty is not so much a physical as a financial one. To store, say, 500,000,000,000 cubic feet of water, equivalent to 11,500,000 acre-feet, would cost, even at the rate of only $5 per acre-foot, $57,500,000. This one fact condemns the project as a system for the exclusive purpose of flood prevention. But whenever such reservoirs have other and more immediate purposes for their construction the increment which each will form in the grand total necessary to produce some influence in the Mississippi floods is an element in its favor worthy of consideration.

"The only direct and effective reservoir project, if any such be possible, for impounding floods of such vast magnitude as those of the Mississippi is that pointed out by Mr. Seddon in the second part of his memoir. The project for utilizing St. Francis basin

* See Report on Ohio Canal Surveys in 1895, House Doc. No. 278, Fifty-fourth Congress, first session, p. 56; printed also in Annual Report of the Chief of Engineers for 1896, part 5, p. 2973 et seq.

for this purpose would only be following out and perfecting the plan upon which nature has operated for an indefinite period. If the overflow into this basin in a great flood like that of 1882 is equivalent to a depth of 6.5 to 7 feet upon its overflowed area of 6706 square miles, it is at least a reasonable question to ask why this flooded area cannot be reduced to one-half or one-third its present size, be given a depth twice or three times as great, and the water be prevented from flowing out until the following low water. The slope of 120 feet in the length of the basin would seem to make possible a division into separate reservoirs by means of moderate embankments such as Mr. Seddon suggests, making five or six basins of an average depth of 10 to 15 feet, with longitudinal levees to restrict the lateral area. The water thus stored (and it could be stored with such an arrangement, whether there were a high flood or only a moderate one) would give to the lower river in low water an increment (based upon the overflow of 1882) of 141,000 cubic feet per second for a period of one hundred days. This would give a low-water flow of at least 300,000 cubic feet per second, and would radically improve the navigation of the Mississippi from Helena to the sea. From Helena up, the slack-water system through the basin itself, with five or six locks, would carry the deep water to Cairo. How far the imperfectly known topography of the St. Francis basin would lend itself to this project, and whether or not the cost would be prohibitory, exhaustive surveys alone can tell.

"On the Missouri River the case with regard to reservoirs is somewhat different. The annual flood of that stream, which is known as the 'June rise,' is essentially a head-water flood. The earlier floods are generally, although not always, from the lower river, and very rarely from the extreme upper sources. The June rise is the mountain flood, bringing down the snow-water, and generally augmented by the spring rains both in the mountains and on the plains below. Not infrequently it meets with heavy contributions all the way down, and is the result of a general high water over all its drainage area. Ordinarily, however, as already stated, it is a head-water flood, and coming as it does while the banks are still soft and yielding from previous high water, it does its full share of the destructive work peculiar to the Missouri River.

"That a complete system of reservoirs in the mountains and plains portion of the watershed of this stream, which should embrace its many tributaries and contain the waters from melting snows and spring rains, would materially reduce the magnitude of the June rise is highly probable. To take off the flood excesses at Sioux City, mentioned by Mr. Seddon in the first section of his memoir, would require a storage of, say, 48,400,000,000 cubic feet, corresponding to a reduction in stage of 2.8 feet. A storage of 100,000,000,000 would probably give the very material reduction of 6 feet. Allowing a reservoir efficiency of only 50 per cent, as elsewhere explained, and assuming that no one of the great floods of the Missouri has its origin in more than one-half of its watershed, it would seem that a reservoir system of 400,000,000,000 cubic feet, distributed over the watershed above Sioux City, would quite effectually control the floods of the river. This amount of storage is about the percentage of total flow required

to be stored for irrigation, as hereafter explained, in order that the water of the arid region may be fully utilized. It must be understood that such a result can be predicted only from a *system* of reservoirs. The effect of any single reservoir would certainly be insignificant, but the combined influence of many might be very important.

"Passing now to the question whether the benefits to the lower river from such a system would be of sufficient importance to justify the construction of reservoirs solely for the purpose of securing them, the answer must be distinctly in the negative. It is still true in this case, as in those already considered, that the benefit is not worth the cost. If, however, there are other and primary considerations, which of themselves would justify the construction of reservoirs, then their influence upon the floods of the lower river is a matter worthy of consideration. And when such primary interests are of a magnitude which looks to a comprehensive system throughout the watershed of the stream, subserving interests of a public as well as a private nature, then the argument for Government assistance in such works stands upon a substantial basis. The point to be especially considered, in connection with such a reservoir system, is that river regulation must always be a secondary motive and more immediate and direct uses the primary motive."

CHAPTER VII.

IMPROVEMENT OF RIVER OUTLETS.

THE improvement of the mouths of rivers is a complicated problem, and its proper consideration would require a volume. It is only intended here to refer briefly to the general aspects of such work.

Formation of Bars.—On reaching the sea the current of a river is suddenly checked in its flow and the sediment held in suspension cannot be carried further. The result is the formation of bars, and these bars generally grow in length out into the sea and rise in height to such an extent that the water, in trying to escape from the river, forces its way through new channels or side outlets of shallow depth. The splitting up of the river into a number of divisions reduces the velocity of the current and in consequence its power to keep open a channel.

There are few rivers entering the sea that are not obstructed by a bar across the mouth, and the permanent removal of this is the object in view when improvements are made. This bar is caused (1) by the deposit of suspended matter carried by the current of the river into the quiet water, and (2) by the action of sea waves. When more material is brought down by the river than can be carried away by the current or tide, improvement by jetties can only be temporary, and recourse must be had to canals or dredging. Where a littoral current exists it is necessary to lead the river out to it by jetties, which contract the waterway and increase the velocity of the current. There is a constant tendency of the sea to throw up bars along the shore, and were it not for the ebb and flow of the tide and the action of the natural currents of rivers the mouths of the latter would become impassable by reason of these barriers. Bars formed by the sea are generally in tidal streams, while those formed by the river are most common in rivers flowing into tideless seas.

Principles Governing Tidal and Non-tidal Outlets.—Vernon-Harcourt has laid down the following principles for improving tidal rivers:*

"(1) The tidal flow should be admitted as far up a river as possible, and all barriers to its progress removed, so that the period of slack water may be reduced to a minimum. By this means also the area of inevitable deposits is enlarged, and thus the deposit does not unduly shoal the channel when the fresh-water discharge is small, and the volume of tidal water flowing through the outlet is thereby increased.

"(2) The fresh-water discharge should not be abstracted, if possible, for supplying canals or for other purposes, but should be directed into the upper end of the main

* Rivers and Canals, p. 235; and Improvement of the Maritime Portion of Rivers.

tidal channel, so that it may have the fullest possible effect in reinforcing the ebb throughout the whole of the tidal course of the river, as the power of the outflowing current to maintain the channel depends upon the additional force thus furnished to the ebb.

"(3) The form of the estuary should be regulated so as to enlarge gradually as it approaches the sea, and thus promote regularity of flow without unduly restricting the tidal capacity above the outlet. This may be sometimes accomplished by low training banks which, whilst directing and concentrating the latter half of the ebb, do not materially impede the admission of the flood-tide up the estuary. Where the estuary is very wide and irregular, and the main river channel through it is very tortuous and shifting, high embankments may be formed, on each side, widening out toward the sea, and the land behind them reclaimed."

The principles governing non-tidal rivers differ to some extent from those given above because of the lack of tidal influences capable of affecting the maintenance of their outlets, and because of the difference in form of the mouths themselves.

In this class of stream the current always flows in the same direction—toward the sea—upon reaching which it is suddenly checked, and, as before stated, deposits its load of sediment. This gradually builds up a bar which forces the water in various directions through separate outlets across the foreshore, and forms what is known as a delta. This division into various arms or outlets tends to reduce the scouring effect of the current, and the channels become too shallow for navigation by reason of the deposit of the matter brought down by the river. These deltas gradually extend into the sea as this material is progressively deposited at the mouths of the outlets.

The following principles are laid down by the authority just quoted for improving non-tidal outlets.*

"(1) The only method of deepening the outlet of sediment-bearing rivers flowing into tideless seas is to prolong one of their delta channels by parallel jetties out to the bar, so that the prolonged current, being concentrated across the bar, may scour a deeper channel, and carry its burden of sediment into deep water further out.

"(2) One of the minor outlets should be selected for improvement, if its delta channel is adequate, or can easily be made adequate for the requirements of navigation; and the discharge of the other outlets should not be interfered with. The advance of the delta at one of the minor outlets is slower, and the distance out to the bar is less, and consequently the jetty works are less costly; whilst an increased discharge, produced by impeding the flow through the other outlets, would also increase the volume of sediment, and therefore quicken the rate of advance of the delta, and hasten the necessity of prolonging the jetties.

"(3) The success of the jetty system depends on a rapid deepening of the sea in front; on the fineness and lightness of the sediment brought down; and on the existence of a littoral current, its velocity, and the depth to which it extends. Any erosive

* Improvement of the Maritime Portion of Rivers.

action of winds and waves along the shores of the delta is favorable to the system, and also any reduction in density of the sea-water, such as may be found in an inland sea.

"(4) If the sea-bottom is flat; if a large proportion of the sediment is dense, so that it is carried along the bed of the river or close to it; if the outlet faces the prevalent winds; and if no littoral current exists, it is possible that an improvement of the outlet may not be practicable; and then recourse must be had to a side canal, starting off from the river some distance up, and entering the sea beyond the influence of the alluvium of the river.

"(5) The bars in front of the outlets of tideless rivers being formed by the deposit from the river, vary in form according to the nature of the sediment brought down. When the material is composed of particles of very variable density, it is gradually sifted as the velocity of the current decreases, and gives a flat sea-slope to the bar. When, on the contrary, most of the material is heavy, the bar has a flat river-slope, as in the first case, formed by the gradual arrest of the sediment rolled along the bottom; but as little of the material is carried beyond the crest of the bar the sea-slope is steep.

"(6) The jetty system does not constitute a permanent improvement, for, sooner or later, in proportion as the physical conditions are unfavorable or the reverse, a bar is formed further out, and a prolongation of the jetties becomes necessary."

The following views relating to jetty and harbor construction on the Gulf of Mexico have been advanced, based on experience with the mouth of the Brazos River, Texas:*

"(1) With tidal harbors the slope of the surface in the jetty channel is inversely as the length of the pass; and consequently, on the Gulf, where the tides are small—about one foot—and the distance to deep water very great, the plan of improving harbor entrances by confining tidal currents between jetties is a somewhat doubtful experiment.

"(2) Jetties, to produce a maximum result at a minimum cost, must be completed beyond the bar in a single season. The bar then acts as a submerged weir with the strongest current on the outer crest, thus transporting all eroded material to a safe distance from the entrance.

"(3) Delays cause great increase in the cost of construction from damage to works by storms, and the much greater distance the jetties have to be built seaward.

"(4) The success of jetty improvements depends largely on the existence of strong littoral currents in front of the harbor entrance; otherwise the advance of foreshore and bar would soon close any channel formed.

"(5) In jetties at the mouths of rivers the strongest currents are at the outer end of the channel, and in no case is it necessary to build the works to a greater depth beyond the bar than that required in the channel.

"(6) When scour takes place in the channel-bed its action is first noticeable at the lower end of the section, and gradually works up stream.

* Transactions Am. Soc. C. E., G. T. Wisner; Patton's Civil Engineering, p. 1312.

"(7) In fresh-water streams flowing directly into the Gulf salt-water currents up stream often exist, where the surface current is flowing seaward; and consequently surface velocities are no sure measure of the discharge of scouring action.

"(8) When the East Jetty of the Brazos River was completed 2000 feet in advance of the West Jetty, the main current at ebb-tide flowed past the end of the unfinished jetty at nearly right angles to the jetty channel. The shoaling which then took place on the bar, and the subsequent deepening when the jetty was extended, very plainly indicate that one jetty would not be very effective for channel-making at ports of this class."

One of the most scientific attempts to study the effects of jetties or training-walls was made by Professor Vernon-Harcourt in 1886, when a relief model, about nine feet long, of the estuary of the Seine was constructed on a scale of 1 to 40,000 horizontal and 1 to 400 vertical.* The bottom of the river was represented by a very fine sand containing a small admixture of peat. The model was tilted back and forth to represent the ebb and flow of the tides, about 25 seconds being the proportionate time for each period. Training-walls made of tin were then placed in accordance with different plans of improvement, and their effects on the movement of the bottom could be clearly traced for the different positions, and the results of theories analyzed accordingly.

It is understood that the German Government has since made use of similar models in the study of the improvement of rivers.

Methods of Improvement. — There are two general methods in use for bettering navigable conditions at the mouths of rivers: (1) by jetties; (2) by canals connecting the deep water of the river with that of the sea. In addition to the results from natural forces channels have frequently to be maintained by dredging. Theoretically, the bar across the mouth may be removed by increasing either the current velocity or the volume of the water. The latter might be accomplished by closing some of the several passes, but this may not be advisable, as all the material brought down would be then deposited at the extremity of the navigable channel. It may be necessary, therefore, to increase the velocity over the bar, and jetties are employed for this purpose. They are located so as to contract the natural section and give direction to the current, but as they do not lessen the quantity of material brought down it is evident that a new bar will be formed farther out unless there are cross-currents to remove it or a great depth of water in which it can be deposited. This will necessitate an extension of the jetties. At the mouth of the Mississippi the crest of the bar was more than two miles at South Pass and five miles at South-west Pass beyond the outlets, and it was necessary in the improvement of the former to carry the jetties out from the shore to remove it.

To quote some examples bearing on the foregoing statements, in the improve-

* Improvement of the Maritime Portion of Rivers, L. F. Vernon-Harcourt, 1892, and Report on Engineering, Paris Exposition, 1889, Wm. Watson, p. 653.

ment of the mouth of the Danube (1858–61), one of the successful examples in this field, the Sulina branch was chosen, although smaller than the other principal one, St. George's, and but little trouble has been caused so far by the deposit of sediment. In this case two jetties were built, and an increase of depth of 11 feet was obtained by 1872, and has since been maintained. The littoral drift being from the north, a gradual building up of the foreshore has taken place under the lee of the southern jetty.

At the mouth of the Rhone two jetties were completed in 1856, leading through one of the main outlets into the Gulf of Foz. This gulf is sheltered from the littoral current, and as a result the great amount of sediment brought down by the river was deposited in quiet water, and the general level of the bottom was raised so that the jetties in later years had to be extended considerably. Finally, owing to the fact that certain coast towns would have been cut off from communication by any further extension, the work had to be stopped with an average depth of only 6½ feet on the bar, quite inadequate to the needs of navigation.*

A striking natural example of the effects of volume and sediment is afforded by the river Amazon, where the principal outlet, discharging enormous quantities of water, is obstructed by constantly shifting mud-banks and shoals, while the Para mouth, with a much smaller discharge, and consequently less sediment and disturbance of the bottom, affords at all times a good channel for navigation.

Where the outlets are crooked and flow over sandy beds, they are usually trained within fixed limits, gradually enlarging into deep water. This increases the tidal flow and facilitates the discharge of the river in such a manner as to render maintenance less difficult.

If properly located, canals solve the problem so far as sediment is concerned, but their locks are a hindrance to navigation. Those canals cut at the outlets of the Rhone, Nile, and Tiber, by the ancients, were mere derivations without gates, and the sediment of the rivers entered at flood-time and bars were formed at the sea extremity. Those of the present day have locks connecting them with the river unless the outlet is so situated that it does not fill with material.

Among the larger tidal rivers there are many whose condition is such that no extensive improvements are required, but there are many others where extensive works for their improvement and maintenance have been installed, not all of which have been wholly successful. In some of these the depths are maintained by dredging. The methods usually adopted for improving this class of streams are those of projecting jetties located at suitable points, or longitudinal jetties following the course of the river. The distance apart these training-banks should be placed has ever been, and is still, a fruitful source of discussion; as well as the height to which they should be built. Vernon-Harcourt states that it is generally unwise to raise the banks above the half-tide level, and that the training of a river by longitudinal banks always conduces to the improvement of an irregular and shifting channel, and gives as instances

* Rivières et Canaux, Guillemain.

of successful works of this kind the Seine, Maas, Clyde, Tyne, Tees, and the Fen Rivers.

On some rivers the training-walls are placed parallel to each other, while on others they gradually converge. Their direction necessarily depends to some extent upon the natural line of the river and the situation of the towns along its banks. The authority just quoted states that the advantage of the system of converging over the system of parallel jetties is, that additional tidal capacity is provided close to the entrance, which promotes the scour through it, and also that deposit tends to take place within the harbor, where it is easily removed, instead of at the actual mouth. The increase in the width of channel between the Seine walls averages 5 feet in 1000 feet; in the Clyde and Tyne 10 feet in 1000 feet; and in the Scheldt 20 feet in 1000 feet.

Materials.—Jetties have been built of masonry, of timber cribs filled with stone, of brush or log mattresses loaded with stone, of piles driven into the river-bed, and connected together by timbers, of piles and stone combined, of fascines secured by piles and weighted with stone, of furnace slag or cinders, of iron cylinders spaced some distance apart, reinforced at their bases by dikes of riprap rising slightly above the water, and of earth protected by stone. The strength, stability, and durability of brush-and-stone and pile jetties in exposed situations is not always sufficient, and more durable materials are required. The log mattresses are rafts composed of trunks about ten inches in diameter, held together by tie-poles about four inches in diameter spiked or bolted to the main timbers. On the top of this structure brush and sawmill slabs are placed in layers to a thickness of twelve to twenty-four inches. When ready, the mats are towed into position and sunk by piling on stone. Sometimes several layers of mattresses are sunk one upon another. Many jetties are built of rubble thrown in loose, while others are made of masonry on a foundation prepared of mattress-work.

Results.—The results realized from jetties have been to a considerable extent disappointing, although in a number of instances the benefits derived from them have been very great. In many cases their failure has been due to a lack of proper location and design, but in the majority of instances the works have been considered experimental from the start, and were built with the hope, not the assurance, that they would prove satisfactory. The problems have frequently been taken up and studied, with the experience gained elsewhere as a guide, and yet the final result has been to disprove many of the theories advanced, and in some cases a final abandonment of the works has taken place and a new start has been made, based on new ideas. There is no doubt of the great value of tidal scour when judiciously directed, but it is a difficult matter to determine in advance the effect any structure placed in a stream will have upon its current and bed. The improvement effected at the mouth of the river Liffey (Dublin harbor) shows how converging piers, arranged so as to receive a large quantity of tidal water within the area they inclose, and discharging it at low water through a contracted opening, can increase the depth at the entrance; but in the Wear, Yare, and Adour, where the works simply guide the outlets into deep water, the increase in depth has

been comparatively small, and it may be doubted whether the prolongation of the jetties would make any material difference. In all doubtful cases it is probable that the employment of a model similar to the one before referred to would prove of very great value in indicating the proper solution of the problem.

It must of course be borne in mind in planning the improvement of a river outlet that designs which would be successful in one case might result in failure in another, and that only a long and careful study of the conditions, supplemented by wide experience, can indicate the best methods to be adopted. Thus at the mouth of the Panuco River, which enters the Gulf of Mexico at Tampico, two parallel jetties were completed in 1892, about 1000 feet apart. The floods in this river are very variable, an interval of three to five years sometimes passing between them, but their violence apparently compensates for their rare occurrence. In 1893 a high rise came, and, confined between the jetties, scoured out over a million cubic yards from the harbor, creating a deep channel.* On the other hand, under different conditions, double jetties sometimes have failed to produce any permanent improvement.

Several examples are to be found, as on the Pacific coast, where single jetties have resulted successfully, with, of course, a large saving in cost. Their theory has been especially developed in America by Professor L. M. Haupt, and described by him in various papers.† Briefly stated, it is based on principles deduced from certain phenomena of the flow of water in channels. Thus in a straight reach of river of alluvial bottom the channel is usually shallow and often variable, while in a bend the same amount of water directed along the curve of the bank will preserve a fixed channel of considerably greater depth. According to the theory, therefore, a single jetty of curved trace, where conditions are suitable for its application, and where it is properly located with regard to littoral drift and other bar-building forces, should produce and maintain a channel where one straight or two straight jetties would produce a shallower and more uncertain one. The term "reaction jetty," or "reaction breakwater," has been applied to this type of jetty, the plan of which, where fully developed, is a reversed or S curve, one part being made concave to the channel at its outer end, the point of reversion being on or near the bar, and the convex part lying inside the bar, and intended to cause a compression or "head" there, and thus concentrate the effluent currents and prevent the deposit of suspended matter. Where the jetty commences at a distance from the shore this feature is intended to arrest at the same time shoaling from littoral drift. The jetty should as a rule be built, as other single jetties, on that side of the entrance from which the sand-drift comes, just as a snow-fence is placed on that side of the road whence the snow comes.

The first known location where this type was tried under the conditions named was at Aransas Pass, Texas, where the concave portion of the curve was constructed in 1895, and was found to have produced, after four years, under conditions of a small

* Transactions Am. Soc. C. E., vol. xlii., p. 504.

† Transactions Am. Soc. C. E., Journals of the Franklin Institute, Proceedings Am. Phil. Society, etc.

tidal variation (14 inches normal), an average increase of depth in the channel of 12 feet.* The cost was about one-third that of double jetties. Another jetty of this type, of small dimensions, was built in 1901 across a dry bar at Longport, New Jersey, for the use of the Pennsylvania Railroad ferries, and is reported to have produced in 18 months a channel from 8 to 12 feet deep.

A curved jetty concave to the channel was also built at the harbor of Swinemuende on the Baltic Sea, many years ago, supplemented by a convex one on the opposite side. It is stated that a considerable improvement resulted in the depth. In this instance dredging is resorted to in order to secure the necessary width.

Mississippi Jetties.—Although a number of river outlets have been improved by the jetty system in this country, the leading example has always been, and will continue to be, the works at the mouth of the Mississippi River. As in some of the more prominent rivers abroad, an attempt was at first made to secure increased depth by harrowing up the bottom so the current might carry away the deposit. This was beneficial to some extent, but not entirely satisfactory, and an attempt was made to combine this method with that of jetties, but it resulted in failure, owing to the lack of stability of the structures and to their meager length. Dredging was next resorted to, and this also proved ineffective for permanent results, and lastly the jetty system was adopted for one of the outlets, the South Pass. This has given fairly satisfactory results up to the present time, but the great increase of commerce and of the draught of ships have shown the necessity for increased facilities, and it is now proposed to open up one of the other outlets, with a channel of greater depth (35 feet). A very complete and interesting history of the various attempts at improving the mouth of this river will be found in the report of the Chief of Engineers for 1899, page 1914. This report shows that not only stirring, jetties, and dredging had been tried, but also that a board had made a favorable report upon the construction of a canal to the Gulf at a cost of ten millions of dollars. This was almost immediately followed by the proposition of James B. Eads (1874), to improve the entrance by double jetties, which, after numerous modifications and delays, was adopted, the South Pass being selected, and the work was completed in 1879. This contemplated 30 feet of depth of channel in the center, with 26 feet of depth for 200 feet in width, the total width between jetties being 1000 feet. It is claimed that the channel required had been maintained between 1879 and 1899 with the exception of about five hundred days. The jetties were made slightly curved in plan, and placed about 1000 feet apart, reduced later where necessary by spur-dikes and inner jetties to about 600 feet. Recent legislation (1902) authorized the improvement of the South-west Pass in order to secure a navigable channel of suitable width with a depth of 35 feet at mean low water. Those having the matter in charge decided upon 1000 feet for the width. This pass is about 15 miles in length, about 13 of which already have a central depth of not less than 40 feet and a width between the

---

* Transactions Am. Soc. C. E., vol. xlii., p. 485 et seq.

35 feet contours of not less than 1000 feet. It is proposed to secure the necessary channel by dredging most of the material, and then to maintain it by two lines of jetties which will not only increase the current sufficiently to prevent silting up, but also protect the channel from unobstructed action of the waves. The jetties will extend to the 20-foot contour on the outer slope of the bar and no attempt will be made to have them parallel, except near the outer end. Their distances apart between centers varies from about 7000 feet to a minimum of 3000 feet. Owing to the fact that the bar will be removed by dredging, it will be possible to maintain the channel with jetties so widely separated, but if the walls were relied upon to do the burden of the work it would be necessary to bring them nearer together. This wide distance apart will permit the placing of the dredged material along the inner faces and thus prevent undermining. The jetties are to be carried only to the height of mean high tide.

In their construction it is proposed to lay flexible foundation mattresses about two feet thick and 150 to 200 feet in width, surmounted by timber grillages two courses high, sunk with rock in the usual manner, throughout the full length of the jetties and for 1000 feet beyond, before any part of the superstructure is started. The latter will consist of a second mattress covered with stone in shallow water, while in depths greater than three feet it will be of concrete blocks or large stones or of concrete laid in place in large bags, possibly containing as much as one hundred tons each. Another method suggested is to build the walls directly upon the grillage in quiet water in monoliths having the full cross-section of the jetty and of a length conveniently handled—20 to 60 feet—and then to sink the grillage on to the mattress foundation, the grillage being held up between barges until the monoliths are completed. The dimensions proposed are a width at the crown equal to the depth of the water at mean high tide, not, however, exceeding nine feet. The side slopes would be made two vertical to one horizontal, and the depth of the block would be four and one-half feet more than the depth of the water at mean high tide, to allow for settlement, less the thickness of the grillage necessary to reduce the weight per square foot on the foundation to the prescribed limit, 300 lbs. The Board having the matter in charge conclude their report * as follows:

"In seeking the solution of the problem before it the Board has given great weight to two important favorable factors, which have not heretofore in similar cases constituted such vital elements of such a problem, namely: The extraordinary progress recently made in the improvement of dredging machinery and the fact that the application of the jetty system to the improvement of the mouth of a very large sediment-bearing stream differs radically from the more familiar case of the application of the same system to the entrance of the ocean harbor obstructed by a sand bar. The first of these favorable factors makes it possible to obtain the desired channel at the South-West Pass by dredging, thereby reducing the function of the jetties to that of merely assisting in the maintenance of the channel. The second factor referred to has made

* Annual Report, Chief of Engineers, U. S. Army, 1900, p. 2297.

it possible to save large sums of money by lowering the height of the jetties, and by utilizing the enormous deposits made by the river to themselves do much of the contraction which would otherwise have to be done by very expensive jetty construction. The one unfavorable factor in the problem, the exceedingly yielding nature of the foundation which the mud flats of the locality afford, has been dealt with by the Board by a systematic effort to reduce the weight of the necessary structures to limits corresponding with the small supporting power of the mud."

PART III.

IMPROVEMENT OF RIVERS BY CANALIZATION.

CHAPTER I.

GENERAL DESIGN AND CONSTRUCTION OF WORKS FOR CREATING SLACKWATER.

General.—By the term canalization, or slackwatering, is meant the creation of a series of pools in a river, connected by locks, and affording at all seasons of the year a depth sufficient for navigation. A few cases are found where such pools are formed by natural reefs or bars of rock, but in the vast majority of instances a dam, which may be either fixed or movable, has to be built to obtain the depth required.

There are certain conditions of existence which are common to all structures used for slackwater navigation, regardless of whether the dams are fixed or movable. These conditions comprise among other things the proper selection of locations, the determination of the level of the sills and of the minimum navigable depth, and the height to be given the lift, or vertical distance from pool to pool.

Where the dam is fixed, its entire structure is brought to the full height to which it is desired to raise the water-level; when movable, the fixed part is merely a foundation on which to erect a suitable superstructure which can be raised or lowered as desired. The lock walls usually are built several feet higher than the crest of the dam in order to enable lockages to be made when the water is considerably above the normal stage of the pool.

Location.—The lift is usually the greatest factor in determining the location of a lock and dam, since the surface of the pool must not only afford the required depth on the sill of the next lock above, but must also provide navigable depth over all obstructions between the two dams. The actual slope of a pool is too uncertain to be taken into account in determining the location, since in seasons of low water it becomes very slight, and is always changing with the discharge of the river. Its flood height, however, must be considered when stationary dams are to be built because of its effect upon adjacent lands and industries, and this is then often of great importance. In movable dams the case is different, because the dam is lowered upon the approach of floods, so that the river-bed has been restored to its natural condition before the water has reached much above the ordinary pool level. For this reason the height of the pools of movable

dams may often be varied to suit the conditions of importance, as for instance, in order to place the works on a desirable foundation, or so as not to divide the harbor of a city into two parts, or for other causes.

In any work for creating slackwater a careful survey of all the available sites should be conducted, and the location made at the place having the greatest advantages. The principal considerations, where the lock and dam are both to be built in the open river and adjoining each other, may be summed up as follows:

1. A good foundation upon which to build the works, deep enough to prevent shoaling below the lock, but not deep enough to make it difficult to reach and expensive to build upon.

2. A straight channel for some distance above and below the lock, so as to provide an easy entrance and exit for boats. This can best be secured by making the location near the middle of a long stretch of straight or nearly straight river.

3. Banks of at least the average height of those along that part of the stream and of firm material, such as clay, rock, etc., which will not be easily overflowed or cut away. This should particularly be the case at the abutment end of the dam.

4. Sufficiently level, or nearly level, land on the lock side of the river to form a yard for purposes of construction, and which will provide sites for the locktenders' dwellings.

Should the location be at a ripple or shoal, then the works should be so built as to make deep water on the shallows; that is, the lock should be below the shoal.

Some engineers prefer a location for a lock "just under the point," that is, just below a bend on the convex side, on the theory that the upper entrance will not fill with drift. Unless the bend be slight, however, a lock so located is difficult of entrance from above and may prove dangerous, as, should a boat become disabled in rounding the point, she might drift over the dam before a rope could be got ashore. The upper entrance also fills with sediment more readily in the quiet water found below a point. Where the bend is very pronounced and the stream narrow it is quite difficult to enter or depart with a tow from a lock so situated.

Other engineers prefer a location directly opposite, that is, on the concave side of a river, in order to reduce the risks to the entrance and exit of craft. This naturally throws the floating drift into the upper approach, but it is to be preferred in some cases to a location on the convex side.

The location of a lock at a bend has the advantage that the river is usually wider at such a point than in a straight reach, and the dam will consequently restrict the waterway to a less degree. Moreover, there is usually deep water along the concave bank, which is favorable to maintaining a good channel. Where the bend is slight, so as to give a slightly curved channel above and below the lock for a distance of several hundred feet, there is but little objection to locating a lock in the concave side of a stream, the tendency of drift and floating bodies to obstruct the upper entrance being really the most objectionable feature.

To illustrate the advantages and drawbacks of point and bend locations we will quote two examples which have come under our notice. In one case the lock had been located just below a sharp point, and towboats entering or leaving the upper entrance, except in low water-season, had to put out lines to trees on the bank in order to escape being carried out into the river and over the dam. Passenger boats usually made a flying entrance, striking broadside on a large timber crib which guarded the upper end of the approach, sometimes succeeding in entering and sometimes being swung out by the current. In the latter case they had to turn in the middle of the river, where there was a strong current toward the dam. This entrance was much troubled by the sediment deposited in the quiet water under the lee of the point.

In another case, the lock had been located in a sharp bend, and boats had similar trouble in entering or leaving, though there was of course much less current driving toward the dam. In certain stages of water, when drift was running, it was swept into the upper entrance and piled up till it lay in a solid mat eight or ten feet deep, and much of it had to be cut into pieces before it could be removed. The deposit of sediment, however, was always slight.

In the earlier constructions and in some later ones, the entire river was dammed at or near the upper part of a sharp bend, and a cut-off or canal dug across the bend from the pool thus formed. In this canal the lock was built, generally near the lower end, to obviate its being filled with sediment from below. Another arrangement, which has been frequently adopted in Europe, is to fix the location where one or more islands divide the river, placing the lock in one arm and the dam in the others. The disposition has generally been to place the dams near the head and the lock near the foot of the island, but some very good works have been arranged the other way. The selection of such a site for a lock and dam, whether at the upper or lower end, or in the middle, and which arm shall contain the lock and which the dam, must be decided in each particular case by a study of the locality and of the conditions affecting it.

In this country the usual practice is to place both the lock and dam in the open stream adjoining each other, but there are cases where a cut-off has been found best suited to the situation. At rapids or long stretches of river or steep slope, it is necessary to build canals connecting the quiet water above and below.

Investigating Site. — In the preliminary surveys necessary to determine a location, soundings and borings should be made at all proposed sites, and after the final one has been agreed upon, the whole area to be covered by the lock, the dam, and the abutment, should be thoroughly gone over with drills, and the depth and character of all materials determined. A dredge is sometimes used to assist in these investigations when the bed-rock is overlaid by a deposit. Test pits or wells should also be put down at suitable points on the banks. Where there is no rock or where its depth is great, borings may be made by driving lengths of pointed gas-pipe, fitting the sections together as they are driven down, or an auger may be used, or in light material a water jet.

To bore into the rock in sedimentary streams, a large pipe may be driven and the sand removed therefrom with a sand-pump. When the pipe has been driven to bed-rock and pumped out, the ordinary drill or diamond drill may be used to ascertain the thickness and character of the stratum. Borings should be made into it at several places for a depth of eight or ten feet, since it frequently happens that a top crust covers a layer of softer material, and unless the presence of the latter be discovered in time, it may lead to annoying if not dangerous results. A sufficient number of holes should be put down to fully determine the profile and character of the foundation. When no bed-rock exists within a reasonable distance or where the material found is unsuitable for a foundation, it will probably be necessary to build on piles, in which case the borings need not be so numerous nor so complete.

After the borings and soundings have been completed, and maps and profiles made, the exact position of the lock and dam may be determined and the plans prepared.

Acquisition of Land.—When the site has been decided upon, the necessary land must be acquired by purchase or by condemnation. This is usually one of the most tedious portions of the work, since titles have to be examined, heirs located, etc., and in many cases the only way to secure a clear title is to have the land condemned. We have met with cases where two or three years have elapsed before proper title deeds could be obtained.

On the lock side, the amount purchased should be sufficient to allow plenty of yard room during construction, as well as good sites for the dwelling-houses. It should extend above the lock about six hundred feet (or more if the bank has to be cut away to widen the approach), and below the lock, unless the banks are very stable and the river not subject to high floods, in a narrow strip of not less than eight hundred to twelve hundred feet in length, since the wash from the dam will attack the bank along that distance.

On the abutment side, it will usually be enough to purchase a width sufficient to extend fifty or a hundred feet back from the top of the bank after the latter has been finished to grade. Above the abutment the property may continue about three hundred feet, and below it, the same distance as on the opposite bank.

In addition to the above, a right of way should be purchased from the lock to the nearest county road, so that communication will always be open.

In the majority of instances, far too little land is secured, as the impression is apparently in vogue that it is unnecessary, or can be purchased later. The result is, either that the neighboring property becomes badly damaged along the river-front, causing perpetual friction with the owners, or that the land has to be purchased after all, involving additional legal expenses and complications in examining the titles, and a higher price charged for land. It would appear to be much the wiser plan to buy a sufficient amount at first, since the extra cost will be a very small percentage of the whole cost of the construction.

All corners should have monuments of stone or concrete marked "U. S.," and the land, if of any value for cultivation, should be fenced.

Arrangement of Lock and Dam. — The question first to be decided is upon which side of the river the lock shall be built. It is not always possible to secure bed-rock upon both sides of the stream. In such cases, other things being equal, the question arises whether to build the lock or the abutment upon the rock. In narrow streams it is usually necessary to place the lock far in the bank in order to avoid restriction of the waterway. If the rock rises out of the water and forms the bank on one side, as is frequently the case, it would be expensive to excavate for the lock and its approaches; but, on the other hand, this rock would form an excellent protection to the bank upon which the abutment is located, since, both with fixed and movable dams, this bank is the one that is most exposed to undermining from the currents.

If the lock is located upon the opposite shore, its foundation must be carried down below danger of undermining; this in some cases will also entail much expense, and a careful comparison alone can show the most economical location.

In movable dams the undermining effect of the water upon the lock foundations is not great, because the pass, which is placed adjacent to the lock, is not opened until the head has been materially reduced by the opening of the weir; but with fixed dams there is a force at work in all rises calculated to disturb the river-bed to considerable depths.

In situations where the foundation is all good or all bad, the lock should be placed upon that side of the river which will afford the easiest ingress and egress to boats.

No fixed rules can be made to suit all cases, and each location must be worked out anew, because ideas which would be applicable to one might lead to unfortunate results in another.

The arrangement of the principal parts being fixed, it is necessary to determine the elevations and general dimensions.

Navigable Depth. — The minimum depth which should be given a system of slackwater improvements is regulated by the draught of the largest craft when loaded, to which must be added six inches or more for clearance. On rivers of small low-water discharge the pools cannot always be maintained at normal height, because of failure of supply during drouth, and on such streams the clearance should be greater than on those where no lowering of the pools takes place. Another consideration, which is too frequently overlooked, is the requirements of the future commerce which may be developed by the slackwater system. A district whose commerce might have been satisfied by a navigable depth of a few feet may become a producer of coal, ore, etc., which will require for economical transportation an 8-foot stage of water, or, if the stream in question is tributary to one having a greater depth, it may be desirable to extend this depth to the tributary in order not to break bulk in transit. It is much better to provide a depth of water too great for immediate wants than one believed to be just sufficient. Commerce will be quick to take advantage of it, and if the dis-

trict possesses any natural wealth the wisdom of the course will soon become apparent.

A valuable lesson in this respect may be drawn from the Seine, where the first system of slackwater between Paris and Rouen was established between 1838 and 1853, at a cost of about $2,800,000, affording a depth of water 5½ feet.* Soon after its completion the improvements in railroad transportation caused serious inroads on the river traffic, and it finally became evident that if the latter were to be kept in existence the system must be enlarged. Accordingly, between 1858 and 1878 new locks and dams were built, and the capacity of the old ones increased to a draught of 6½ feet. The cost of these changes was about $2,800,000. The relief, however, was only temporary, and between 1878 and 1888 a further expenditure of $12,200,000 was incurred, securing a depth of water of 10½ feet, and the result has been the creation of an immense and increasing traffic on the river, and a general development of the valley.

A similar example has been afforded by the St. Mary's Falls Canal, in Michigan, between Lakes Superior and Huron, through which passes most of the interstate commerce of the Great Lakes. The first canal with its lock was completed in 1855 at a cost of about a million dollars. In 1870 enlargements had to be commenced, which were finished in 1881, and cost over two million dollars. These soon proved insufficient, and in 1887 more extensive work was begun, including a lock 800 feet long and 100 feet wide, which was completed in 1896 and provided a depth on the sills of 21 feet. The cost of the last improvements was $3,700,000 but it is nevertheless becoming evident that further accommodation must be provided before many years.

Similar demands are being made at other points in this country, and it is becoming apparent that if river traffic is to hold its ground many of the existing systems must be enlarged. At present most of our new locks are designed for a depth of six feet on the sills, but it would be much better to provide for eight or nine feet.

Size of Lock. — It may be taken as an axiom for similar reasons, that it is better to build a lock too large for present needs, than only large enough. The trend of modern transportation is toward cheap rates, which means larger boats and greater draught of water, and if rivers are to hold their own in competition with railroads, it will be necessary to improve them with this in view. A railroad must renew its rails and rolling stock because of wear, and can then make them suitable to modern demands, but a lock once built can only be changed by a special and very heavy expense, which could have been avoided at the outset for a small part of the cost.

We are acquainted with one instance of a small lock which proved of practically little value, since the only boats which could utilize it had to be of small tonnage, and could not carry much freight, and the expenses were consequently too high in proportion to the profits. Several boats attempted to establish a trade, but all met with the

* Canalisation de la Seine, Boulé, 1889.

same causes of failure, and as a result there is now no traffic on the river except that of timber. The lock in question was 27 feet wide in the chamber, and of a length sufficient to contain one barge.

The widths of chamber used on rivers in this country are usually 27 feet, 36 feet, and 52 or 55 feet, and on the Ohio, 110 feet. These widths are based on the size of a standard coal barge, which at present is 25 to 26 feet wide, and 125 to 135 feet long. A lock of the first size should not be built, unless under very special circumstances, since it is too small to accommodate steamboat traffic, and if another size would be too large for the river the project should be abandoned. It is much better wherever practicable to build to the width of 55 feet, with a good depth of water on the sills, even if existing locks on the same river are smaller. It should be remembered that the extra width merely involves some additional excavation, and a little extra expense for miter walls and gates, while if the chamber is built too small, it must either be enlarged later at serious expense, with delays to navigation, or it will not properly fulfill the purposes for which it was designed.

A few old locks have been enlarged in this country; but where new ones have been built, even on streams having small locks, larger ones have frequently been put in. In France many locks have been torn down and rebuilt with greater capacity.

The practice of building the chambers with entrances narrower than the basin has not obtained in America, although it is often to be met with in Europe. Thus on certain of the new locks on the Moldau the entrances are 36 feet wide and the basins 65½ feet wide. This saves material in the gates and cross-walls, and also avoids much excavation of the approaches, but it delays the handling of the traffic, as the boats, if in tows, have to be moved sideways in the chamber.

Large locks, on the other hand, while they have advantages, have the inconvenience of wasting water and consuming time in filling and emptying the chamber where only a small lockage is to be made. Generally this waste of water is of little consequence, but there are times on some rivers where the supply of water is insufficient and must be saved as far as practicable. An intermediate pair of gates is sometimes introduced, and this not only obviates the objections named above, but has also the additional advantage of allowing the continuous use of the lock during repairs to the other gates. Another method, is to build a small lock adjoining the large one, of a size to accommodate single boats and the smaller tows. The locks at Bougival, finished in 1883, and at Suresnes, near Paris, are of this type. Where a fixed dam is to be constructed adjoining the lock, the smaller rivers are not usually sufficiently wide to admit a double lock, but in wide streams and where movable dams are to be built this plan has many advantages.

Lift.—In determining the lift of a lock and dam, that is, the vertical distance to the crest of the proposed dam from the crest of the next one below, due regard must be had to the riparian property and to any adjacent water-power mills or industries. The heights of bridges crossing the stream may sometimes have to be consid-

ered also. If navigation alone is to be consulted then the fewer the dams the better, within certain limits, because each is an obstruction which causes more or less delay to transportation, beside expense for maintenance, and by increasing the lifts in establishing a system of slackwater the number of locks and dams can be reduced. It must be seen to, however, that the sizes of the parts to be maneuvered are well within the power available, and the effect which the flow of a great volume of water will have on the works themselves and on the banks below, with the consequent danger from undermining or cutting around, must also be considered. The effects upon the régime of the river, in causing shoals or bars which may become obstructions, must not be omitted from the study. With proper care in the selection of the locations, and proper study in the design of the various parts of the works, these objections can usually be overcome.

In France, where practically all the dams are movable, the lift is generally small, although in the later dams it has been increased considerably. In this country, where nearly all the dams are of the fixed type, the lifts are rarely less than ten feet and in some cases as much as eighteen feet. Where the banks along the stream are high and of firm character, there is no objection to making the lift high, provided proper precautions are taken in the construction. With movable dams the lifts are usually small in this country also, rarely being over eight feet.

Height of Lock Walls.—The height of the walls above the crest of the dam should be such that but little time will elapse between the time they are overflowed and the time when boats can pass over the dam. In France they are usually built to the level of the highest navigable water, and this rule is followed on some streams in this country where navigation ceases at a moderately high stage, but it is not applicable to our larger rivers where boats run at stages of fifty or more feet above low water.

With movable dams, the coping is usually placed four to six feet above the upper pool; with fixed dams it is made from nine to twelve feet, except where the flood range of the river is very small, when it is placed lower. The object of having a good height of wall—technically known as the guard-wall, or guard—above the upper pool, where the dam is fixed, is to allow the river to rise to a considerable height before it can drown the lock and thus interfere with navigation. Theoretically, when this occurs, boats should be able to pass over the dam, but practically it is a condition not always realizable except at an expense quite incommensurate with the benefits to be secured, since it may require an inordinate height of wall. This is especially the case with rivers from the mountains, where the rises are very rapid. However, in such streams, by the time the locks are drowned, the current has usually become too swift and dangerous to allow boats to run, and, at the most, navigation is only suspended at "drowning-out" stages for a few days each year.

In some constructions the lock walls are built a few feet lower at the lower end than at the upper end, for the purpose of cheapening construction.

When the water-way has been restricted in the construction of the dam, it is

necessary to have a higher guard than where a long spillway has been provided; otherwise, the river will "go out of lock" much sooner than it should.

Foundations. — The natural foundation is a factor of the first importance in determining the lift, and, therefore, the general design of the lock and dam. On a rock foundation, with other conditions favorable, a lift of extreme height can be used, while if the foundation be light or porous, it is a serious risk to employ other than low lifts, especially with movable dams.

Wherever rock exists within a reasonable depth, the construction should always be commenced on it, as the extra expense will be repaid by the general stability of the work. If the rock be found to be soft, it should be removed till a hard stratum is reached, or be covered with a good bed of concrete wherever exposed to the action of the water.

The foundation is the most important part of the entire work, as if it is improperly designed or constructed, it can rarely be remedied without rebuilding, and any defects will invariably become manifest, though sometimes not for many years. The action of water is so penetrating and insidious that it will attack the slightest weakness, and unless the engineer can detect and secure during construction the vulnerable points of the foundation and of the banks around the structure, trouble is sure to result. This is especially true of foundations on seamy rock, as the head of water when the chamber is full will force an escape through every crevice that has been overlooked, and if the rock be soft it will be steadily eaten out. For this reason it may be best with a doubtful foundation to cover the entire floor with concrete, and thus avoid uncertainties.

Where the natural foundation is other than rock, the masonry may be commenced directly upon the surface, if the latter be sufficiently hard. Where this is not the case, piles are usually employed, and in foundations of this character it is well to drive continuous sheet-piling along the three outer sides of the lock, to prevent any escape of the material by undermining. Sheet-piling must of course be driven along the upper side in any case to cut off the upper pool, and where the soil is very porous a second row is sometimes driven above the upper miter wall.

Where piles are used it is the general practice in this country to cap them with $12'' \times 12''$ to $12'' \times 16''$ timbers, to which a heavy timber floor is spiked, on which the masonry is commenced. It is much better, however, to excavate the soil for a foot or two below the pile-heads, and commence a bed of concrete on it, several feet in thickness, inclosing the piles. When this is done the weight of the wall is largely carried by the surface of the natural foundation, instead of by the piles alone, and greater stability secured against settlement. In the former style of foundation the entire load is transmitted to the piles through the caps, and it will generally be found on investigation that the pressure per square inch between the surfaces of contact is greater than usually considered safe, especially as wood immersed in water and always under pressure appears to become gradually soft. It is probably owing to the crush-

ing of weak timbers from this cause that walls on pile foundation have sometimes cracked in two.

Lock and dam masonry should not be built on a foundation one part of which is composed of piles or grillage and another part of the natural material alone, because unequal settlement will take place and crack the walls, if not at first, then later on when the timber has had time to show its defects. We have seen an instance of this kind in a lock wall which did not show any settlement until after the lock had been in use for some years, although since the first break no additional settlement has been apparent.

Coffer-dams.—A tight coffer-dam is very desirable for hydraulic foundations, especially where construction is to be commenced on the bed-rock. There are two general styles employed, one known as the pile coffer, the other as the crib coffer. Another style, known as the box coffer, and made of plank, has sometimes been used in light construction. The first is much the cheaper, but is not as suitable as the crib coffer for many localities. It is composed of a row of round piles, 8 to 10 feet apart, to which 10″ × 10″ or 12″ × 12″ walings are bolted. On the outside a close or continuous row of sheet-piling is driven to the rock, or as far down as it is desired to go. The outside is then well banked with good clay, or clay and gravel, and where necessary riprapped against scour. Sometimes a second row of round piles is driven inside, to which the outer piles are braced, but the additional support afforded is of questionable value, since if the excavation draws out material from around them, as is almost always the case, they have little bracing power against the water.

Another variety of the same coffer consists of two similar rows of piles, with a space of 8 or 10 feet between, and tied together with 1-inch or 1¼-inch tie-rods and struts. Walings and sheet-piling are provided for each row, placed on the inside, and the vacant space is filled with clay. The outside is banked, and riprapped where required. This type costs from $10 to $16 per foot run of coffer, according to circumstances.

These coffer-dams require a sufficient natural bed in which to drive the piling, and where this exists they have been successfully used for heads of water of 15 to 18 feet, even in light material, and they will withstand rises excellently. They require plenty of space, however, since they must be set well away from the masonry in order to prevent loss of support during excavation by the removal of the inside supporting earth.

The second type is the crib coffer, and consists of logs or sawed timbers, spiked upon each other, with sets of ties 10 or 12 feet apart. The inside faces are planked with 1-inch boards to keep the filling in, and the pens are filled up with clay and riprapped on the top, and well banked or "backed" on the outside. Sometimes, with a porous material, a row of sheet-piling is driven along the outside, but if plenty of backing is used this is not necessary. This style is much more expensive than the pile coffer, since it requires excavation and more material, but it must be used in cases where there is no bed to hold piles, or where the contraction of the river might cause

their undermining. It has the advantage, however, of permitting excavation to be carried close up to the cribs. These are usually made with a width of base not less than 12 or 14 feet.

The box coffer consists of horizontal waling-pieces, supported by temporary uprights, and placed 3 to 4 feet apart, inside which planks of 1½″ to 2″ in thickness are placed vertically. Heavy tie-rods pass through the wales to prevent spreading. The box thus made is filled with gravel, clay, etc. The planks should be driven slightly into the river-bed.

It will be found sufficient in most cases to place the top of the coffer-dam from 6 to 8 feet above low water. When the river has risen to a height of 6 or 7 feet, the leakage usually becomes equal to the capacity of the pumps, and there is risk, moreover, of part of the filling being washed out, but the additional height above this elevation is a safeguard against the overflow of the coffer before it has become filled through the sluices. These should always be provided at the down-stream end, so that the pit can be flooded when required They can be closed with needles or stop-planks.

It is desirable in many cases to design the coffer so that it can be opened and entered by a dredge early in the season, in order to facilitate excavation of deposit from floods, or of the bank. This can be done without difficulty in a pile coffer, and in a crib coffer the opening can be arranged for with sheet-piling, or with a bank of good material well riprapped.

Where repairs to a lock or dam are to be made and completed in one season a serviceable coffer-dam may be obtained by throwing up a bank of earth around the space to be inclosed, and protecting it with riprap where exposed to scour. This is a very useful method where repairs are to be made to a fixed dam, as it obviates any necessity of drawing off the pool and stopping navigation. We have used it where an entire dam had to be torn out to the bed-rock, and rebuilt, a section at a time, during a season of many and unexpected rises. The crest of the coffer had to be raised several times to prevent the river from overflowing it, and after the work was finished it was finally eaten into at its ends by a flood which rose to a height of 4 feet above the crest of the completed dam.

Materials of Construction. — Modern locks and the foundation of movable dams are almost always built of masonry. Where placed in a derivation, on a river or canal of no great importance, the lock has sometimes been formed by simply placing masonry ends to the chamber, and grading and paving the banks between to form the lock-pit. Wooden locks of modern construction are to be found on the Fox River in Wisconsin, where the extreme flood range is only about 3 feet, and serve their purpose excellently. They have masonry ends, and the chamber is formed by upright posts covered with two thicknesses of 2″ dressed plank, jointed but not calked, behind which a dry rubble wall is built. The cost of these locks is about one-third that of masonry locks, but they have to be renewed about every ten years.

For locks exposed to high floods, however, masonry should always be used, as its mass is necessary for endurance. The choice of materials for it is limited usually to sandstone, limestone, or concrete. Limestone is usually preferable to sandstone, as it does not soften and wear so easily under the action of the water.* Concrete is very largely used at the present time, although experience with its durability in river works has been short. There seems to be no reason, however, why it should not prove satisfactory. For works exposed to the action of a river it is best to use a concrete of Portland cement, at least above water. The difference between the cost of Portland and natural cements at the present day is small, and the former will give a harder and better concrete in all particulars. The higher grades of natural cement have given excellent results in foundations, but the cheap grades, of which there are many on the market, have not proved satisfactory for important work.

The use of timber for permanent construction, wherever it will be exposed to the action of moving water, should never be permitted. The accepted belief that timber below water lasts forever is only partly true, for in order to do so it must be protected from all currents. This fact was discovered long ago by European engineers, but in America we find many examples of wooden lock floors, wooden foundations, and other vital parts which the water will sooner or later wear away. The sheathing of a dam bears strong witness to the rapid wearing force of water; and while a lock floor is exposed to much slighter currents, the effect is just as sure. We have seen wooden lift-walls, exposed simply to leakage from the gates, become dangerously worn in twenty years, and have removed timbers from the heart of a dam, where very little water could have reached them, which were worn and channeled half away. Wherever it can possibly be done, nothing but masonry should be exposed to the action of the water.

Protection of Banks.—This will be found described under "Fixed Dams (Abutment)."

Clearing the Pool.—After the location has been selected, the river above should be cleared of bowlders, snags, and timber throughout that part which will be affected by the new pool. It is best to commence this work at once, so the débris will disappear before construction is finished, and also so that steamboats may be able to utilize the stream in moderate stages of water as soon as the obstructions have been removed.

If it is not expected to complete the lock and dam for some years, the trees can be merely deadened, or "ringed." When this is done, the trees will gradually die and fall, a branch at a time, into the river, and will be safely carried out by the floods. Some trees, depending on size and species, will die and begin to disappear in eighteen months, while others, especially sycamores, will struggle for life from three to six years. Where they are below the level of the new pool, they should be deadened close to the roots, so no stump will be left when the tree falls.

* The softer classes of limestone, especially the oolitic varieties, have not proved durable in the face-work of lock masonry, as after a few years' exposure to the weather the stones begin to scale off. For a similar reason no machine-finished surface and no bush-hammered surface should be used in exposed work, as the action of the tools deadens the surface and causes it to scale.

If it is desirable to remove the timber at once, the trees must be deprived of their branches, and the trunks cut into lengths of ten or fifteen feet, so they will pass out of the river without forming snags. As with deadened timber, they must be cut off close to the ground, so there will be no hidden obstructions left. Above the level of the new pool it will only be necessary to remove the timber that overhangs the river, and in wide streams even this is not required. Where the trees are large, cross-cut saws are more economical for their removal than axes.

Dangerous stumps, if the ground is hard, can be removed by Judson powder, a mixture of black powder and dynamite, and which must always be fired with exploders or percussion-caps. It is very useful also in removing broken or creviced rock, as it can be poured into the cracks, thus saving drilling.

Any dangerous bowlders or reefs should be blasted out, if possible to eight or ten feet below the new pool level. For this it is best to use a high grade of dynamite, as it will shatter the rock more and make its removal easier.

The cost of clearing a pool, where the timber is heavy and reefs and bowlders plentiful, may reach as high as $300 per mile.

General Remarks on Design. — In designing the various parts of a lock and dam, the engineer should bear in mind that repairs to works of this nature are always costly and tedious, not only because the presence of water has to be contended with, but also because the lock or dam may have to be placed out of operation, thus interrupting navigation. For this reason especial attention should be given to making all parts as simple as possible, and in addition, if the parts are connected with the operation, such as valves, pintles, etc., they should be so designed that they can be easily removed and replaced when worn or broken. This is a matter which is unfortunately too often overlooked, and forms one of the unnecessary difficulties to be contended with in repairs to existing structures.

The same principles apply to movable dams with still more importance. In the design of works of this class, where material of temporary life must be employed, such as wood or iron, a large excess of strength should be provided, not only because wear and rust will weaken them, but because they are subject to blows and twisting from drift and ice, and occasionally from craft, the forces of which must always remain indeterminate. As far as experience in this country has shown, the life of the trestles and wickets of a Chanoine dam appears to be about twenty years. In that time the water will practically wear out the timber of the wickets and will eat up the ironwork of the trestles, and the latter process appears considerably more active on steel than on iron. It has not been found practicable to protect such ironwork adequately with paint, as the scour of sediment and the long immersion soon damage and undermine the coating, and once an entrance has been gained the rusting will continue beneath slowly and insidiously, and will create a surface so rough that it can never be thoroughly protected again.

All journals, pins, or other movable parts which are to be subject to immersion

should have a play of at least $\frac{1}{32}$ of an inch in their holes, or they will become so rusted in a few months that their removal, or operation where they act as hinges, will be very difficult.

Provision should be made for renewing the anchor-bolts of lock and dam-sills, etc., where they are embedded in the masonry, since their exposed ends will sooner or later rust away. This may be done by putting sleeve-nuts or turnbuckles on them near the top. It will then only be necessary for renewal to excavate to the turnbuckle and to screw in a new end for the bolt.

Where several locks and dams are to be built, a uniform design should be adopted as far as practicable for all similar parts, so that the machinery of one lock will be interchangeable with that of another. This facilitates not only the work of construction, but also the work of repairing any parts that become worn or broken.

After the works have been completed, all dèbris should be removed and the grounds should be laid out, graded, and planted with grass and trees. Lock-tenders, with a little encouragement, will take pride in keeping their premises trim and in good order, and although it may require some extra expense to secure these results it will be found that it will be very small, and hardly worth considering in view of the general advantages gained. Moreover, in works of such magnitude and under the charge of the Government, it may justly be claimed that they should be completed and cared for in all respects with a high degree of excellence.

CHAPTER II.

LOCKS.

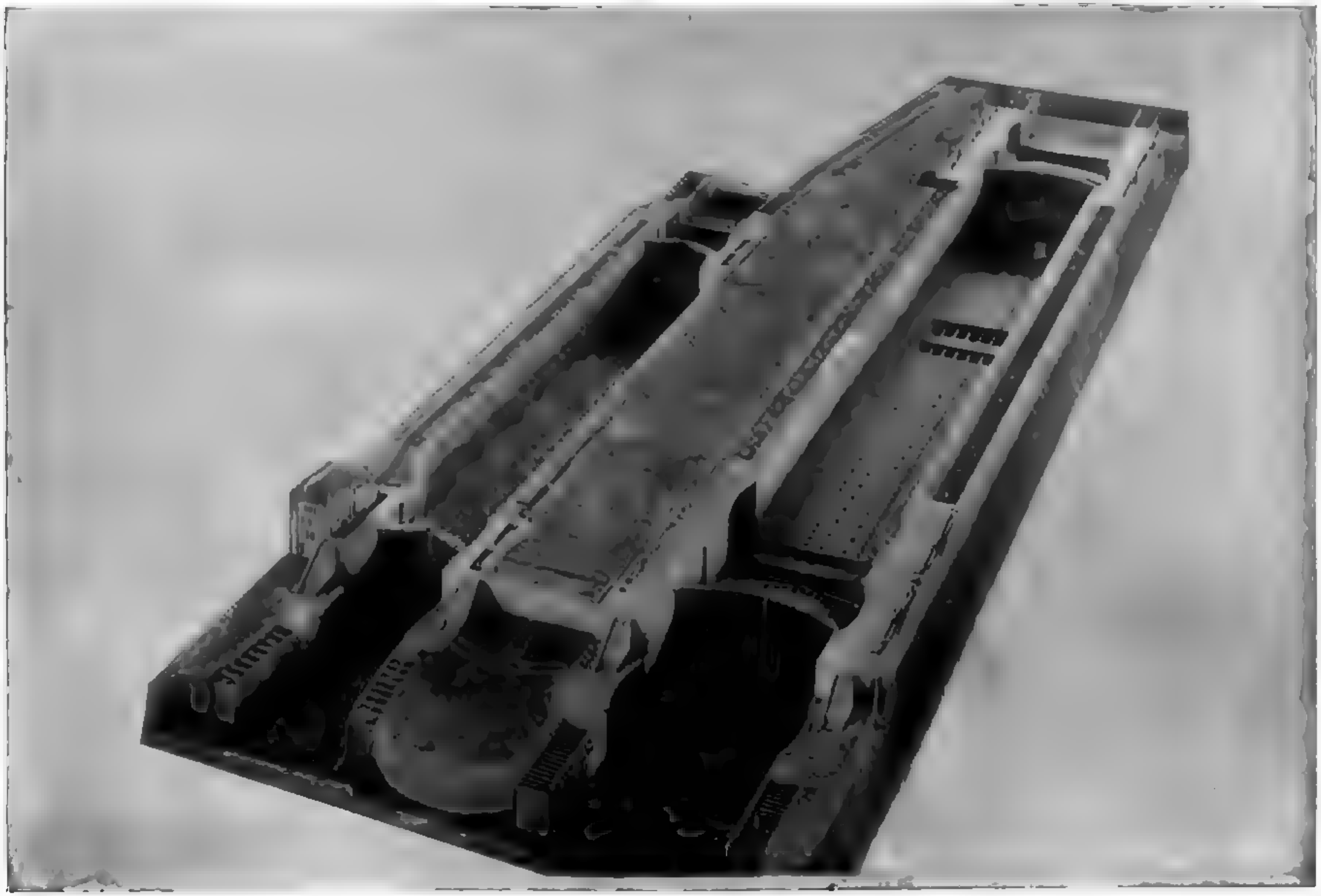
Origin.—The date of the invention of locks is somewhat uncertain. By some they are ascribed to the Dutch, and are said to have been originated in 1253; by others they are claimed for Leonardo da Vinci, while still others say that Philip Visconti was the inventor. In the "Annales des Ponts et Chaussées" for 1847 it is stated that the first one was built to facilitate the transport of marble for the Milan cathedral, and Lombardini, an Italian engineer, says this was done by Visconti in 1439 to connect the old and new lakes, the difference of level being about 10 feet, and claims further that da Vinci could not have made use of locks until 1460.

Description.—A lock consists of a rectangular basin called "the chamber," having side walls of masonry, earth, or timber, connected near the ends by gates of wood or metal. Through the chamber thus formed communication is established between two pools of different level by admitting water from the upper pool through conduits and discharging it into the lower pool at the lower end.

That part of the lock above the upper gates is called the head bay or fore bay, and is flanked by the head-bay walls, and that below the lower gates the tail bay, flanked by the tail-bay walls. Between the tail-bay walls is the lower coffer-wall, used for coffering the chamber. The gates close at the bottom against sills inclined up stream and called, from their position and construction, the upper and lower miter-sills. The lower miter-sill is generally fastened to a wall connecting the side walls of the lock and called the lower miter-wall, while the upper miter-sill is attached to the lift- or upper miter-wall; this wall also connects the main walls of the lock and is sometimes called the breast-wall. The upper miter-wall frequently contains the filling culverts. The head-bay walls are connected by a cross-wall, sometimes also called the breast-wall, but generally known as the upper coffer-wall. Of the two side walls the inner one is known as the land-wall and the outer as the river-wall. The land-wall generally has wings at its ends extending into the bank. Gate recesses are formed in each wall just above the miter-sills, into which the gates swing when opened, out of the way of passing craft. The recesses are terminated at the lower ends by stones or castings of special construction, known as hollow quoins, into the hollow of which the gate fits when shut. At the up-stream end of the recess is a right angle, round or beveled on its outer face, called a square quoin.

NOTE.—Certain of the illustrations in this and the following chapters are reprinted by permission from "The United States' Public Works' Guide and Register," by Captain W. M. Black, Corps of Engineers, U. S. A.

The first lock ever built on the American continent was erected at the Soo. In the year 1790 a lock 38 feet long, 8 feet wide, with a lift of 9 feet and a draught of 2½ feet, was built. It was used by the fur-traders for lifting their heavily laden birch-bark canoes to the level of Lake Superior. This lock has been carefully preserved.



MODEL OF THE LOCKS AT ST. MARY'S FALLS CANAL, MICHIGAN.

The small lock was finished in 1881, and the large one in 1896. A portion of the floor of the latter is removed to show the culverts.

(To face p. 146)

Maneuvers.— Boats are let into the lock when the water therein is at the level of the pool from which they approach. To pass a boat through a lock ***up stream*** the upper gates are closed, the lower ones opened to admit the boat and then closed again; and the water is let in and fills the chamber, lifting the boat to the level of the upper pool. The upper gates are then opened and the boat proceeds. If a boat desires to pass down stream, the lock being full to the upper level and the gates being opened, the boat enters, the upper gates are closed, the water is let out of the chamber till it is at lower pool level; the lower gates are then opened and the boat can go out.

Calculations for Lock Walls. — In determining the proportions of lock walls, there are usually three main parts which govern the general dimensions. These are:

(1) The river wall of the chamber.

(2) The land wall of the chamber.

(3) The mass required in the head-bay and tail-bay walls.

There are minor portions which must also be examined, such as the wing walls, which frequently act as retaining-walls, the miter-walls, which are usually made in arch form, the upper coffer-wall or wall across the head of the lock, etc.

River Wall of Chamber.—The forces acting on the river wall are the weight of the masonry and the pressures of the pools; but in order to be secure against the most unfavorable conditions, the lower pool should be assumed as drawn off. This will then leave only the pressure from the upper pool acting on the wall. With fixed dams, which are usually placed opposite the head walls, the maximum will occur with the chamber full, but with movable dams, which are usually placed toward the lower end of the lock, the maximum will occur with the chamber empty, the pressure being in this case on the outside of the wall.

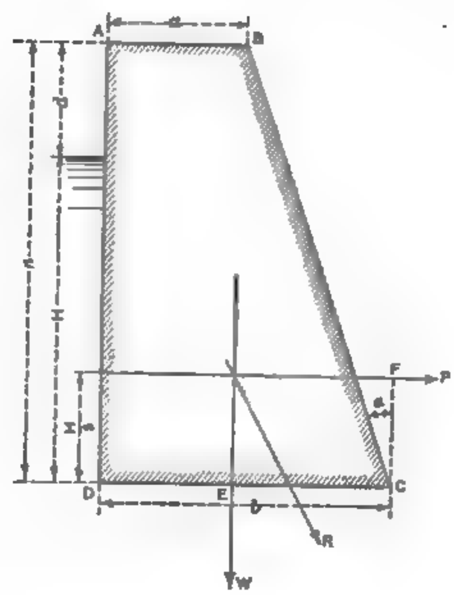


Fig. 1.

Where the wall is built on solid rock there occurs no loss of weight by immersion, since the water cannot penetrate under the foundation and cause upward pressure. The full weight of the masonry is thus available for stability. It is the practice, however, of many engineers to take no account of this in their calculations, so as to secure an additional margin of safety, while others assume a mean of the two, and suppose the wall to lose 31¼ lbs. of weight per cubic foot. Unless the rock is known to be dense and solid, too much reliance should not be placed on its imperviousness. Where the wall is on gravel or any other porous material the full loss of weight of course occurs.

The top or coping width may be made from 5 to 6 feet, as this will give sufficient width for the maneuvering without excess of masonry.

Let $ABCD$ (Fig. 1) represent the section of a river wall; a = width of top; b = width of base; h = height; H = head of water; d = distance from top of wall to water; w = weight of masonry per cubic foot.

The weight W of the wall per running foot is

$$w\left\{ha + (b - a)\frac{h}{2}\right\} = \frac{wh}{2}(a + b).$$

If the lower pool is supposed to act, with loss of weight by immersion, the weight of the wall will be reduced by the area of the submerged portion multiplied by $62\frac{1}{2}$ lbs.

The pressure P on the vertical face per running foot is

$$H \times \frac{H}{2} \times 62\tfrac{1}{2} \text{ lbs.} = \frac{H^2}{2} \times 62\tfrac{1}{2} \text{ lbs.}$$

The factor of safety is found by dividing the moments of the resisting forces by the moments of the overturning forces, taken about the point of rotation. Thus the factor is

$$\frac{W \times EC}{P \times FC}.$$

If P act on the sloping face BC, its value will be $\frac{H^2}{2} \sec \alpha \times 62\frac{1}{2}$ lbs., and moments will be taken about D. The factor of safety should be not less than 2.5, and is usually made larger.

The calculation may be made graphically by drawing the forces and finding the position of the resultant R. Its direction should be such that it will cut the base-line at a distance from the edge never less than one-fifth of CD, and the best practice requires that it should fall within the middle third, in accordance with the well-known rule for walls supporting pressure. In certain cases this will show a factor of safety greater than 2.5.

The width of the base can of course be increased by offsets, if found necessary.

Where there is likely to be much difference between the pools when the lock is "drowned out," as mentioned in the calculation for lock-gates, it may be needful to examine the upper section of the wall, to see if it has mass enough to resist the head. An increase, however, will be rarely, if ever, required if the coping is of the usual width.

Land Wall of Chamber.—This portion of the lock walls is usually calculated for two conditions, one with the upper pool filling the chamber, the lower pool drawn off, and no filling behind the wall, and the other with the pressure of filling behind the wall and no water in the chamber—a condition which occurs when the pit is pumped out for repairs. For the latter case it is usual to assume a full lower pool pressing behind the wall, so as to be secure against the most unfavorable circumstances. If the section found necessary for this is as large as that given to the river wall, calculation for the

first condition will be unnecessary, since assumptions are identical. An exception to this might occur with movable dams of high lift, as the water outside the chamber presses on the sloping face of the river wall, while on the land wall it would press on a vertical face.

Let $ABCD$ (Fig. 2) represent the proposed section of the wall, W its total weight per foot run, T the earth-pressure on the back of the wall per foot, which may be

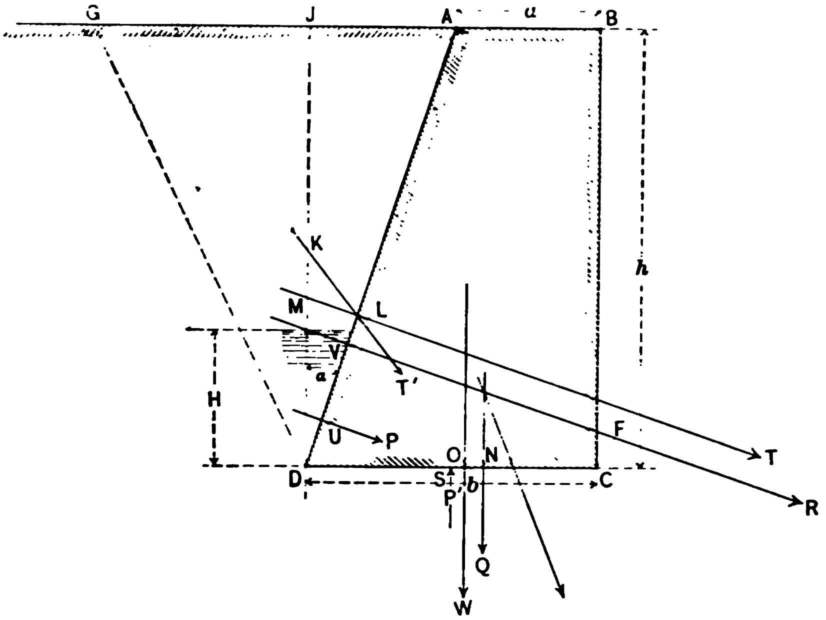


FIG. 2.

assumed as acting normally to it at a height equal to $\frac{h}{3}$, and P the pressure per foot from the lower pool, supposed to have seeped behind the wall. The other letters are the same as before.

If w = the weight of the masonry per cubic foot,

$$W = \frac{wh}{2}(a + b).$$

If the wall is on a porous foundation, we have an upward pressure on DC from the lower pool, since it extends under the floor of the lock as well as behind the wall. This pressure per foot run is $bH \times 62\frac{1}{2} = P'$, and acts at the center of DC.

The pressure P is

$$H \times \sec \alpha \times \frac{H}{2} \times 62\tfrac{1}{2} \text{ lbs.} = \frac{H^2}{2} \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.}$$

The pressure T of the earth, if the earth is assumed as level with the coping, and without friction on the back of the wall, is

$$\frac{\text{Weight of triangle } AGD \times GJ}{DJ}.$$

If it is desired to include the effect of the friction of the earth, as is customary, the direction of T is changed to T' (the point of application remaining at $\frac{h}{3}$), and T' equals

$$\text{Weight of triangle } AGD \times 0.643.$$

This is a formula given by Trautwine, and has the advantage of simplicity, besides giving results as practical as those of more complicated formulas. The line DG is inclined to the vertical at half the angle of repose of the backing.

The angle KLM of inclination of T' to the back of the wall is equal to the angle of wall friction, and is given for this formula as about 33° 41′, or 1½ to 1, LM being perpendicular to AD.

If the back of the wall is vertical, the triangle AGD becomes the same as the triangle JGD.

If the normal pressure T is used, we must first find the resultant of T and P, which equals $T + P = R$, and acts at a point V which may be found by equating the moments of P and R about L.

Thus

$$P \times UL = R \times VL, \quad \text{or} \quad VL = \frac{P \times UL}{R}.$$

Combining next W and P' (if the latter exists) into their resultant $W - P'$, or Q, the factor of safety becomes

$$\frac{Q \times CN}{R \times FC}.$$

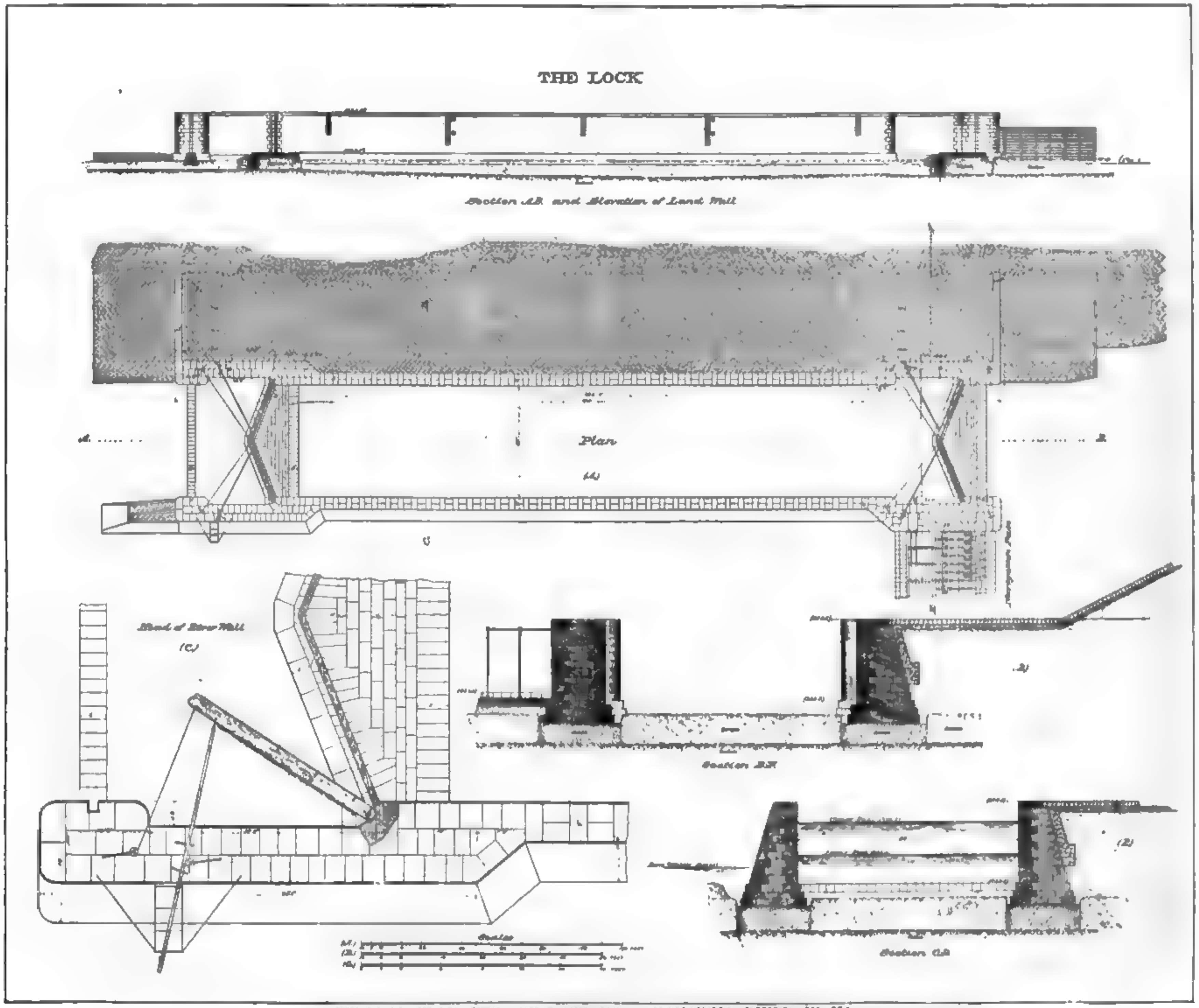
The force Q, if P' and W do not pass through the same point, acts on that side of W opposite to P' at a point N, found by equating the moments of W and Q about S. Thus

$$Q \times SN = W \times OS, \quad \text{or} \quad SN = \frac{W \times OS}{Q}.$$

The result may be checked graphically, as in the case of the calculation for the river wall.

If the inclined pressure T' is used instead of T, the resultant of P and T', which will pass through the point of intersection of these two forces, can be found by graphics. As it will usually fall within CD, it will be found most convenient to combine it graphically with Q, limiting the position of the final resultant through the base CD as before indicated for the river wall.

The assumption of an inclined earth pressure, as might be expected, affords an apparently greater stability of wall than the assumption of a normal one.



GENERAL PLAN, ETC., OF LOCK NO. 7, KANAWHA RIVER, W. VA.

(To face p. 150.)

If the wall is surcharged (see Fig. 3), a case which is occasionally met with in practice, the triangle AGD becomes $AG'D$, $G'D$ being, as before, inclined to the vertical at half the angle of repose of the backing, and the pressure T'' becomes the weight of $AG'D$.

Then, if we include the friction on the wall,

$$T'' = \text{weight of } AG'D \times 0.643,$$

the angle of inclination KLM being as before, about 33° 41′, or $1\frac{1}{2}$ to 1.

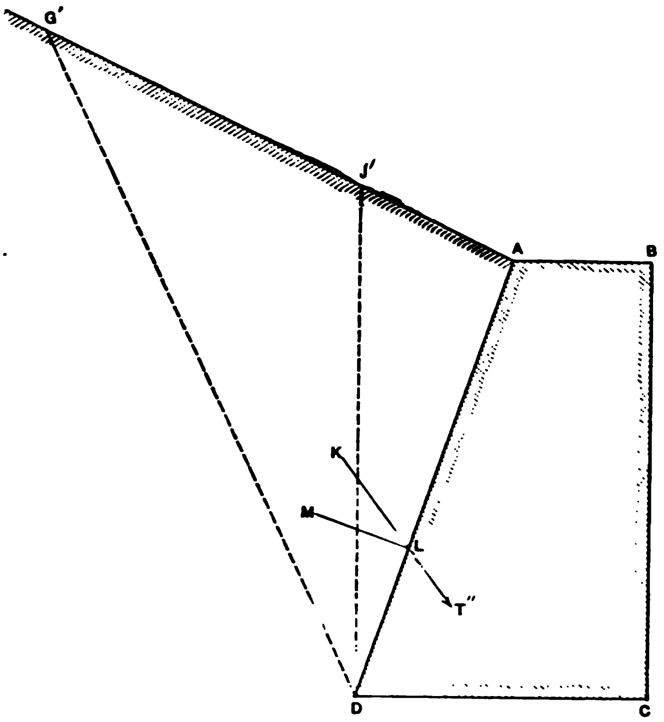


FIG. 3.

If the slope of the earth does not extend high enough to make $AG'D$ a complete triangle (see Figs. 3 and 4), $AG'D$ becomes a four-sided figure, as $ADG''G'''$, and its weight is reduced accordingly. In this case the point of application of T'' is not at one-third of the height of the wall above the base, but at that point where a line QL', drawn through the center of gravity of the earth mass and made parallel to the slope DG'' of maximum pressure, strikes AD. This point will lie more than one-third of the height above the base.

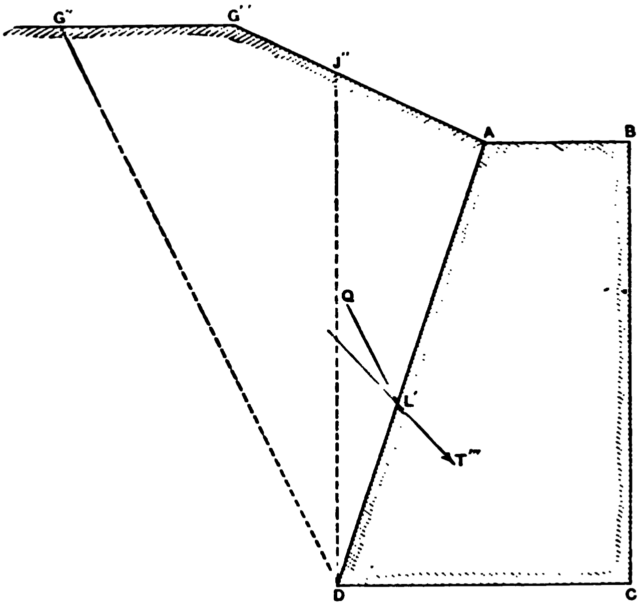


FIG. 4.

Head- and Tail-bay Walls.—The functions of the head- and tail- bay walls are, first, to support the concentrated pressure from the gates, and secondly, to provide width for maneuvering the operating machinery, such as the capstans for opening and closing the gates. The latter condition usually requires a width of at least 10 feet behind the gate recess.

In providing against the thrust from the gates it may be assumed that the masonry for a certain length, say 15 feet, above the hollow quoins, acts as a monolith with that below them. Thus, let the accompanying Figs. 5 and 6 represent the head and tail walls on the river side in plan and section, the

masses $ABCD$ and $FGHJ$ being taken as those which resist the pressure from the gates.

The forces acting are the weight of these masses, W and W', and the resultant thrusts, T and T', from the gates. Let Q be the pressure on the leaf and its miter-wall, α the angle of inclination of the gates, H the depth of water on the floor of the lock,* l the half width of the chamber, and e the depth of the gate recess. The surfac. FK is subject to a pressure P from the upper pool in the chamber, but that on AE, if E is above the crest of the dam, is balanced by the equal pressure on the outside of the wall.

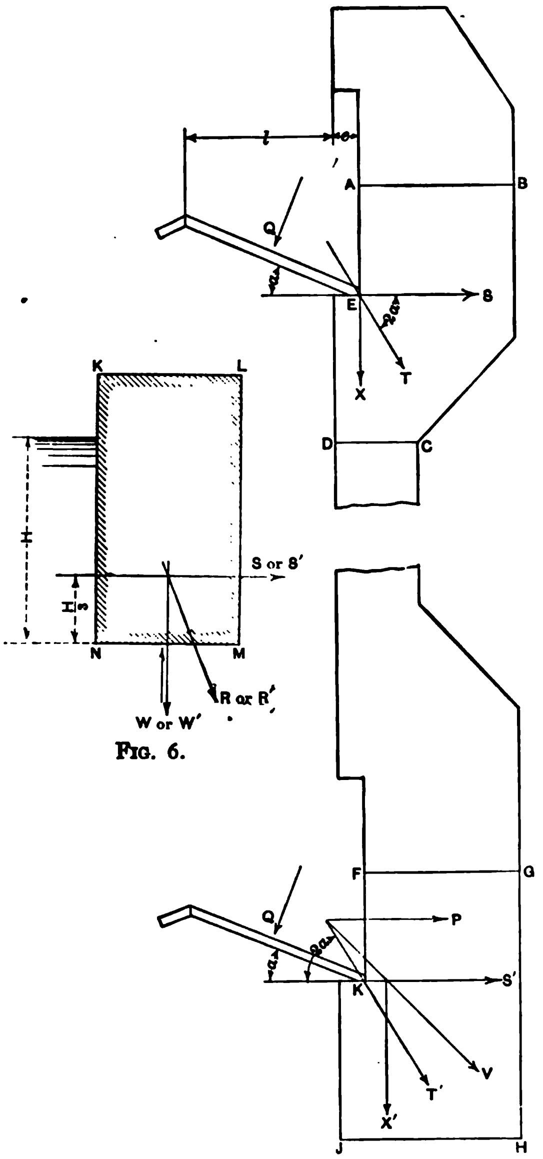


FIG. 6.

FIG. 5.

The maximum of T occurs at E when the chamber is pumped out for repairs, and the upper pool is full, or more than full; the maximum of T' at K when the upper pool is in the chamber and the lower pool is drawn off. In the case of $ABCD$, if the crest of the dam is below CD, there will be pressure from the upper pool on the outside of the wall, which we will neglect here, but which makes for safety. If the foundation is porous, there will be an upward pressure on the base from the lower pool, which must be deducted from W. The upward pressure from the upper pool is supposed to be cut off by sheet-piling. The pressure of the lower pool on the outside of the wall will be neglected, as it is small.

T and T' should be calculated for the head of the water on the floor of the lock, since the pressure below the gates on the miter-wall is transmitted to the main walls, and they have thus to support the entire thrust. We then have, as found in the calculations for lock gates,

$$T \text{ or } T' = \frac{Q}{2 \sin \alpha}.$$

* Where the miter-wall is high, it usually possesses sufficient mass to resist the pressure of water against it without any support from the main walls. In such a case H is measured from the bottom of the gate instead of from the floor.

Also,

$$Q = \frac{H^2}{2} \cdot \frac{l+e}{\cos \alpha} \times 62\tfrac{1}{2} \text{ lbs.}$$

The angle which T makes with a normal to the axis of the lock will be found equal to 2α, the angle α itself being usually made about 20°.

At E resolve T into two forces, one acting along the wall, the other at right angles to it. The former equals

$$R \sin 2\alpha = \frac{H^2}{2}(l + e) \times 62\tfrac{1}{2} \text{ lbs.} = X.$$

As it is supported by the chamber wall it need not be further considered. The force at right angles equals

$$R \cos 2\alpha = \frac{H^2}{2} \cdot \frac{l+e}{\tan 2\alpha} \times 62\tfrac{1}{2} \text{ lbs.} = S.$$

It acts at a distance above the point of measurement for H equal to $\frac{H}{3}$, and combined with the weight of the wall W will give us the factor of safety, which may be found and checked as shown for the river wall. The width of base MN may be taken as being the same as the width of the base of the wall behind the gate recess, and the center of gravity of the mass $ABCD$ as passing through the center line of this base. This will simplify the calculations, and the error resulting will be on the side of safety.

The total length of wall which is assumed to act as a monolith in resisting the force S should not be over 30 feet. If this provides an insufficient mass, the base width should be increased.

The calculations for the lower end of the wall are similar to the foregoing, but in this case the force P must be combined with T' to give the resultant horizontal pressure V. The shortest way will be to do this graphically, and V can then be resolved into a transverse force S' and a longitudinal force X'. Combining S' with W' we can find the position of R' as before, and combining X' with W' we can find the position of the resultant of the longitudinal component, which must of course fall within $FGHJ$ at a distance from HJ sufficient to avoid undue pressure on the masonry and the foundations. This component, as will be readily seen, cannot be neglected as was X, since there is no chamber wall below to support it.

The strains on the head- and tail-bay walls on the land side are the same as those on the river wall, and the masses are usually made the same, or slightly less. The earth pressure is, of course, an element of safety, as it opposes the thrust of the gates, and if it is desired to take it into account, the forces can be combined as in the preceding calculations.

If the walls are of concrete, which is usually built up in blocks with vertical

joints from bottom to top, these should be arranged so as to avoid a joint near the hollow quoin, and to place behind the gate a monolith at least equal in mass to that required by the calculations. The foundations of the lower end of the walls must be designed with ample strength to resist force X', as, if any weakness exists, the gates will tend to separate the concrete blocks at the nearest joint, a case of which we have met with. This will sometimes occur in masonry locks also, several such cases being on record. One of them, which occurred in this country, is worth noting because of the unusual circumstances. The lock, which was built about 1840, carried a maximum head of some 15 feet. The lower end of the river wall, where the parting occurred, was built on squared timbers placed on bed-rock, a peculiar method of construction of which several examples are to be found on the same river. The mass of the masonry appeared ample, but the wall had parted in an irregular seam across the gate recess and moved visibly as the water in the chamber rose or fell. The cause lay in the timber footing, which had been worn away where exposed to the eddies of the current. The wall was satisfactorily repaired by building a concrete buttress on the outside.

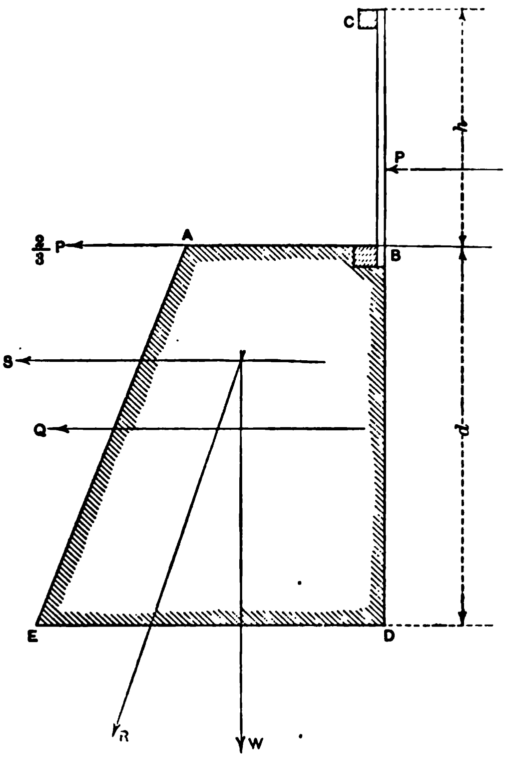


FIG. 7.

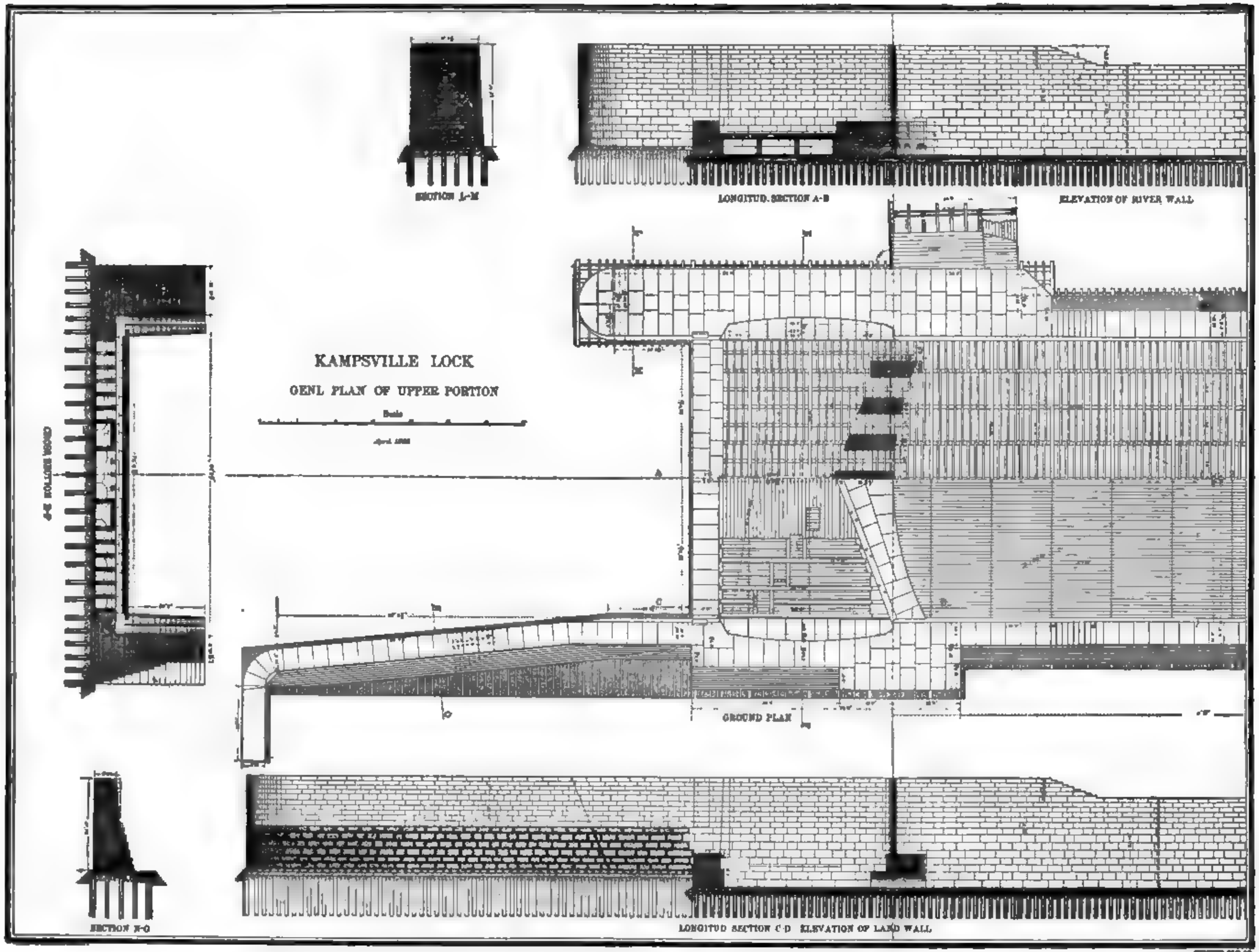
Upper Coffer-wall.—The upper coffer-wall is only used when the lock is to be repaired. Its top is provided with a wooden sill which supports the needles used for the coffer-dam, their heads being supported against a beam just above the upper pool.

Thus, let $ABDE$ (Fig. 7) be a section of a wall supporting needles CB. Let h be the head of water on the needles, and d the height of the wall. The pressure on CB per foot run, supposing the needles to be vertical, is $P = \frac{h^2}{2} \times 62\frac{1}{2}$ lbs., of which two-thirds go to B. The pressure on BD per foot run, which may be taken for the sake of simplicity as acting at the center of BD, is

$$Q = d\left(h + \frac{d}{2}\right) \times 62\tfrac{1}{2} \text{ lbs.}$$

Combining Q and the proportion of P at B into their resultant S, and finding the weight of the wall W, the position of the final resultant R may be determined as shown for the river chamber wall.

The space behind AE usually remains filled by water when the head bay is pumped



GENERAL PLAN, ETC., OF THE UPPER END OF THE KAMPSVILLE LOCK, ILLINOIS RIVER, ILL.

(To face p. 155.)

out; but as it may be necessary at some time to empty it, the pressure on AE should not be relied on.

Elevation and Angle of Miter-sills.—The lower sill is usually placed at the depth below pool required by navigation; thus, if a depth of 6 feet is to be provided, the top of the sill is placed 6 feet below pool. It should never be less, as in a dry season, with little water running, the entire pool will closely approximate the level of the crest of the dam below, and if the latter is leaky the depth on the sill will be accordingly reduced. For this reason the sill should be placed as low as practicable, since an excess of draught is much better than too little.

This principle is generally adopted for the upper miter-sill where the dam is fixed, and if a 6-foot depth is desired, for example, the sill is frequently placed from 8 to 12 feet below the upper pool. Should the river have to be improved at any future date, so as to afford a greater navigable depth, a sill so placed will not have to be disturbed, and a large saving will have resulted with no greater first cost.

The sills of the upper and lower coffer-walls are placed at the same elevations as the upper and lower miter-sills respectively.

Where the lock is connected with a movable dam the upper and lower sills are generally placed at, or nearly on, one level, and from 6 inches to 3 feet below the sill of the pass, depending on the navigable depth required and other circumstances. By this arrangement the lock gates can be left open when the dam is lowered for the winter, preventing the chamber from filling with deposit, and at the same time affording more area of discharge.

The economical angle of the sills, or the angle which permits the minimum of material in the gates, will be usually found to lie between 19° and 21° with a normal to the face of the walls. For this reason an angle of 20° is frequently adopted.

DETAILS OF CONSTRUCTION.

Floor.—The floor of the chamber, where artificial, is a most important part of the construction. Any weakness which it may develop can only be repaired by pumping out the pit and stopping navigation entirely, and in extreme cases it may be necessary to build a coffer-dam around the entire lock in order to cut off the outside water. Such repairing, moreover, is always tedious and expensive, and it is difficult to make it thoroughly satisfactory.

The floor is subject to the downward pressure of the water when the chamber is filled, and to upward pressure from the lower pool when it is pumped out, and also, judging from experience, to the pressure from the upper pool in many cases as well. As it is impossible entirely to cut off leakage from the upper pool through sheet-piling, stone drains have sometimes been laid under floors to allow the water to escape. It may be questioned whether this remedy would be permanent, as in sediment-bearing rivers the drains would probably soon become filled with mud, and our own experience

has led to the belief that it is best to make the floor strong enough to resist the upward pressure, and to let the seepage find its own escape.

Too much reliance should not be placed on the ability of sheet-piling to stop seepage. Where it can be driven under the best conditions, as was done at the Wachusetts Dam, Mass., where each pile was planed and sunk through soft material with water-jets,* it may prove very effective; but as these conditions cannot be secured on river-work, the chief function of the piling appears to be to obstruct the carrying off or the flow of material. To quote an example: on the upper end of the coffer-dam for a new lock, triple-lap sheet-piling was driven to rock through 2 to 3 feet of sand and mud, the depth of water on the rock being about 13 feet. The piles, which thus required a minimum of driving, were of good lumber, rough but not warped, and were carefully put down. In spite of the favorable conditions, however, when the slope of the river caused a head of about 6 inches between the upper and lower sides of the piling, a copious leakage was visible everywhere through the joints, and this continued until the backing was completed.

The usual method of constructing an artificial floor in this country is to bolt timbers to the sides of piles, covering them with planking or squared timbers. These should always be calked, unless a layer of concrete is placed over them, as the water will force its way through the smallest opening and gradually enlarge it, and if the foundations are of light material, it may, in time, cause undermining. The calked timbers should be covered with planking spiked down so as to prevent the oakum from being forced up. It is far better, however, to discard timber altogether, for reasons mentioned elsewhere, or else to cover it with a layer of concrete bedded around spikes driven into the wood.

A type of floor which is in favor in Europe, and which is probably the best of any in use, consists of an inverted arch of cut stone or of concrete, the voussoirs in the former case being cut either for a flat or for a curved arch. The expense of the method has probably prevented its wider adoption, but with concrete the cost would be little if any more than that of a flat floor, and the gain in strength would be very great. In the locks of Dinant and Anseremme, on the Belgian Meuse, where the chambers are 39 feet wide, the floors were formed of inverted arches of 2 feet rise, the voussoirs being 15 inches deep, and laid on a bed of masonry of a least thickness of 1¾ feet. Below this was a bed of concrete 2 feet thick. At the new Suresnes lock on the Seine the rise was made 20 inches in 59 feet, the arch being laid on a solid bed of masonry.

In one or two instances in America the floor has been placed 10 or 12 feet below the lower miter-sill, and surrounded on all sides with concrete, exactly like a box without a lid. This box is then filled with gravel or other heavy material, and covered with plank spiked to joists, the idea being that the weight on the floor will coun-

* Proceedings Am. Soc. C. E., April, 1902, Frederic P. Stearns.

teract the upward pressure. It may be questioned, however, whether this method offers any advantages in cheapness or efficacy over that of the inverted arch.

An example of the force of the upward pressure occurred on one of the tributaries of the Ohio, some years ago, where a lock floor, composed of piles and concrete, broke up suddenly, the rupture taking place as a large hole, approximately 60 feet square, at the upper end of the chamber. The lock was in use at the time, and as attempts to pump out proved ineffectual the hole was temporarily filled with gravel till final repairs could be made. A careful examination showed that the accident was due to pressure from the upper pool, which broke under the upper miter-wall and ruptured the floor. To cut off this pressure a large amount of backing was placed above and outside the lock, which has been in use since that time without further repairs, the hole apparently allowing only a small amount of water to pass in and out.

Other instances are on record of floors being broken through or partly displaced, and no pains should be spared to make this portion of the structure thoroughly secure.

A space of 30 to 60 feet below the lock, when the foundation is not rock, should be protected with an apron crib or heavy riprap, so that no washout will be caused by the discharge from the valves.

Where conditions permit, it would be well to provide a "sump" or drainage basin in the floor, for the use of the suction when the chamber has to be pumped out, as the water could then be drawn entirely off the floor, instead of remaining on it to the depth of a foot or more, as is usually the case.

Battered Chamber Walls.—A very desirable method of construction is to build the chamber walls with a batter on the inner face of $\frac{3}{8}$ or $\frac{1}{2}$ inch to the foot. Few examples of this are to be found in America, where the walls are almost invariably vertical, but it is doubtful if any engineer who has had experience with both types has failed to see the advantage of a battered face. On a certain tributary of the Ohio, examples exist of each kind, and where the vertical faces have become seamed and scarred with the rubbings of craft, the battered faces show hardly a scratch after twenty years of use. The reason is of course that with the latter the bottom of a barge will strike the wall first, and whereas the top of the barge is protected with iron and has protruding bolt-heads which will cut and scar the masonry, the bottom has only the edges of the wooden planking with which to strike. The additional expense of construction for battered walls is very small.

It would be preferable, however, to continue the batter from end to end of the walls, instead of making the head and tail walls plumb, as is the usual practice, since, with this combination method of building, the vertical walls become badly scarred by craft entering or leaving, and the corners of the walls and quoins become chipped off.

The width of a chamber with battered walls is of course measured at the bottom so as to give the full dimensions required.

Miter-sills.—These sills are usually made of $12'' \times 12''$ timbers, with their tops a foot or eighteen inches above the floor. In the older locks they are frequently 16 to

18 inches square and 2 feet high or more above the floor. Their object is to provide an elastic cushion for the bottom of the gates. They should be well bolted down, since they are sometimes subjected to a lifting pressure from the gates, and when once started the upward water-pressure is of course added. This has occurred at the locks of the St. Mary's Falls Canal, at those of the Louisville and Portland Canal, and elsewhere. The sills are also subject to blows from the gates in closing, and sometimes, where the dams are leaky and the water low, to the strain of boats being dragged over them.

They should not be lap-jointed, but should be so constructed that any piece can be taken out and replaced without difficulty, and without disturbing another piece.

Miter- and Coffer-walls.—Where the miter-walls or the coffer-walls are shallow, and consequently of small mass, they should be well secured to the foundations with bolts or otherwise, or the water may leak beneath and force them up, an accident of which several examples have occurred in this country.

Culverts.—See paragraphs on "Valves" in the next chapter.

Quoins.—In locks of cut stone the hollow quoin is cut directly in the pieces, which are set on each other as the building progresses. Especial care must be taken to see that they are set perfectly plumb and level, or it may be necessary to trim the whole quoin after all the stones are in place. The gate bears directly against the surface of these stones, which are made concave to suit the radius of the heel.

With concrete locks the quoins have usually been made of cast iron, 1 inch or 1¼ inches thick, in sections 5 or 6 feet long, planed for the gate-bearing surfaces, and bolted to each other and to the walls. This kind requires as much care in setting as those of cut stone, and a more satisfactory method, as shown by recent practice, is to shape the quoin directly in the concrete, using a form in the way employed for any other special surface. This is cheaper than using cast-iron sections, which are expensive, and also gives a better alignment, as it is difficult to keep the iron sections in exact position. If the shop-fitting on them has been at all defective, additional difficulties will be encountered.

The best shape for ordinary locks consists of an arc of the same radius as the heel of the gate, and in length about one-half of the semicircle. A flat quoin has been used on the Osage River, in Missouri, placed normal to the resultant thrust from the heel, but it is understood that the ordinary circular quoin is to be substituted in any new locks. This should be placed in relation to the heel so that very little space will exist between, or sticks and other floating débris, such as is always present, may get behind and strain the gate in closing.

Wing Walls and Drain.—The upper and lower ends of the land wall are usually provided with wing walls, running into the bank from 30 to 60 feet from the chamber face, with the object of preventing the water from cutting round behind the walls. Where the upper wall does not rest on rock, sheet-piling should always be driven along the up-stream side, but with the lower wall this is not necessary. They

Rear View of a Lower Land Guide Crib, Ordinary Type.

(To face p. 159)

View of a Lower Guide Crib Formed of Separate Cribs with Grillage Timbers.

View of an Upper Guide Wall Formed of Timbers Spiked to Piles Replacing an Old Crib.

frequently have to act as retaining walls also, and should be designed accordingly. Sometimes they are joined to the bank by a timber crib. In this case the latter should be sheathed inside and filled with tamped clay. Riprap should never be used to fill it.

Where no drain is placed behind the chamber wall the upper wing wall can be made shorter, and in some cases it has been omitted altogether, although the advisability of this is doubtful. The practice of putting in such drains is one which certain experienced river engineers have strongly condemned, and apparently with excellent reason. They are placed there for the same purpose that they are placed behind retaining walls, to carry off any water that may have collected. Usually they are made of riprap, and are 2 or 3 feet in width, extending from the bottom of the wall to near the top, and from the lower end, where they connect with the river, to near the upper end. It is thus seen that the back of the wall is open to the pool below, and that the only obstacle to prevent the upper pool from flowing round is the length of the upper wing wall. We have met with one example where the upper pool in high water forced its way around, passing under the paving to the drain, fortunately without causing serious damage. If the drain be omitted, the water must force its way through a mass of earth two or three times longer, and the gain in safety would appear to be worth more than the doubtful utility of putting in a drain which the river will sooner or later choke with sediment. It may be added that the land wall in any case is usually designed to support the earth under the conditions of greatest pressure, that is, without any relieving drain.

If one is put in, it should always discharge directly into the lower pool and not into the tail bay, since, if the latter arrangement is adopted, it will not be possible to pump out the lock.

Paving.—The space behind the land wall and between the wing walls is usually paved, to prevent floods from washing out the backing. This paving may be composed of small blocks, 10 or 12 inches deep, set on edge, or of large stones laid flat, or of concrete. If the first style is used, it is a good plan to grout the joints with cement, or there will be a constant growth of grass and weeds in them. Where large stones are used they are difficult to reset if any sinking occurs, and for this reason are not desirable.

With a new lock the paving should never be laid till the filling has been in place for one or two years, otherwise it may become badly disfigured by settlement. Temporary protection can be secured by the use of riprap.

Coffer-dams.—Arrangements for closing the lock at each end for repairs should always be provided. A method long in use for this purpose consists in placing timber beams, one on the other, across the head and tail bays. The ends of these timbers rest in vertical recesses cut in the walls, and they are supported at intermediate points against posts connecting with and braced to the masonry. A needle-dam of the Poirée type is also used for the same purpose, and is preferable in many ways, as it is simpler and more easily handled, besides requiring less timber. Where this type is

used a single beam is employed for the top support, its ends resting in horizontal slots in the main walls, and the needles rest against this and against a wooden sill in the upper coffer-wall. The slots should be placed about 2 feet above pool level, so the beam can be put in at any ordinary stage of water. For very small locks a simple 12″ × 12″ timber will suffice; for locks of 36 feet in width the timber must be trussed or provided with a middle strut and a tie-rod, while for locks of wider opening the timber usually rests on movable iron trestles hinged to the coffer-wall. In this case it may be necessary to provide counterforts or buttresses from that wall, to support the down-stream ends of the trestles. A steel beam may be used instead of a wooden one, if so desired.

Another, and in many ways a better method, is to omit the trestles, which sometimes rust away before they are needed, and to span the opening with a horizontal truss supported on temporary posts, and designed so it can be taken apart into two or more sections for easy handling. Trestles also possess the disadvantage of liability to injury when the lock entrance has to be dredged.

Guide Cribs or Walls.—The upper and lower entrances to a lock should be provided with at least one guide crib each, to assist boats in entering or leaving, and to which barges can be tied while waiting their turn to lock. They should be flush with the chamber face where they meet the lock walls and may either continue in the same line or flare outward from it. Where the lock is used by towboats the former alignment is preferable for the land side, as the fleet can then be pushed straight into the chamber. The length of the cribs should be not less than the length of a tow which can pass through at one lockage.

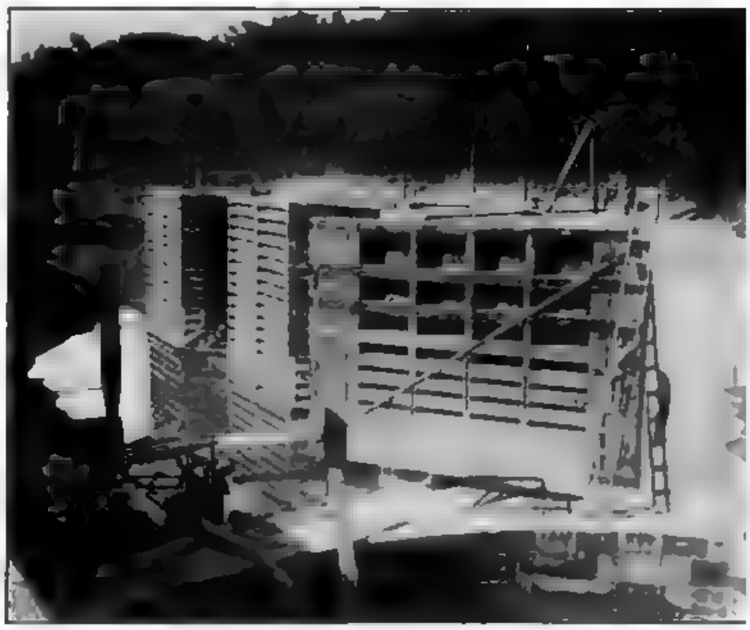
In Europe such structures are built of cut stone or of concrete, and in this country the example is now being followed gradually, the entire wall being made of masonry, or of masonry under water, surmounted above water by a wooden crib filled with riprap. Where wood is used a very convenient size for the timbers is 10 inches square, with sets of ties 10 feet apart, and spiked together with a drift bolt at each intersection. Sometimes these timbers are dapped or notched into each other, but experience has shown that equally good results are obtained by simply spiking the ties and stringers upon each other, with butt joints where the latter come end to end. In putting in the stone, large pieces should be picked out and set with flat faces on edge against the openings between the timbers at the front and end of the crib, as this will give a much neater appearance than if the stone are left as they are dumped, and the extra cost is very small.

The life of these structures is from ten to fifteen years. A frequent cause of failure, especially where the cribs are high, lies in want of care in providing for the weight of the filling. Actually, a very large portion of the riprap is upheld by the timber, since its projections catch in the spaces between the ties or stringers. After a year or so the whole crib becomes choked with sediment, and the result is that an almost solid mass of masonry has to be carried by the ends of the ties, and this is where

View of a Triangular Upper Guard Crib (River Side), with Connecting Bridges.

(To face p. 161.)

General View of the Grounds of a Lock.



View of an Upper Lock-gate of Timber, with Triangular Guard Cribs.

the first signs of failure are always shown. We have seen ties split open in new cribs through this pressure, and in old cribs they will be found to have become rotten at the ends, while the adjoining timbers show little decay. Relief may be obtained by placing blocks on each side of the ties to distribute the load, and care must be taken that the foundations are also equal to the weight to be upheld.

The tops of the cribs are usually made 6 to 10 feet wide, and the width of base not less than one-half the height. As the vertical wedge of the filling always tends to push the crib out at the top, the face should be battered ¼ inch or ⅜ inch to the foot; this may be done by setting each timber back as the building progresses, and we have found it to prove very effective in retarding the forward settlement of a crib. Diagonal bracing is also useful in this respect. Where high cribs are built with vertical faces, and no provision made against undue weight upon the timbers, they will speedily begin to settle, and in a few years may lean over 12 inches or more at the top. In many cases the end of the crib next the lock wall has been bolted to the masonry, with a view to retarding this settlement. The practice, however, is of very little use, and results in unsightliness, as the end is held up while the rest of the crib settles, and the difference in level becomes more noticeable each year. We have always found that where a timber guide crib or a dam of timber cribs is of any height, a settlement commences as soon as the work is finished, owing to the weight of the filling gradually compressing the fibers of the wood, and the best that can be done is to so design the work that the settlement will be equalized as far as possible.

The tops should be made level with the coping of the chamber wall, so that boats can use them until the river has flooded the lock.

For the upper entrance the cribs are usually placed on both sides, the river crib being needed to keep tows from being drawn toward the dam. In this case a short crib is usually sufficient on the land side, serving to keep craft from striking the wall. Where the bottom is good, a cheap and effective guide may be made with piles driven deeply 8 to 10 feet apart, and provided on the lock side above the upper pool with a grillage of 10″ × 10″ timbers, spaced 10″ apart vertically, and bolted or drift-bolted to the piles, thus forming a continuous crib face. Some of the piles may be left projecting above the top timber, to serve as check-posts. The outer end should be provided with a short crib or a cluster of piles, as a buffer-post. In other cases where this method would cause trouble to barges because of the water drawing toward the dam through the open spaces, rectangular or triangular cribs 20 feet to 30 feet in length may be employed, spaced 20 to 30 feet apart, the openings being spanned by a grillage of timbers as for the piles. These types of guides will usually be found preferable to solid cribs, as the entrance does not so easily silt up, and they are less costly in establishment and repairs, while proving as satisfactory to boats.

A plain row of single piles, or of clusters of piles, is not desirable unless the entrance is an unusually easy one, as the corners of barges catch against them, delaying the maneuvers.

If a solid crib is used, an opening about 15 feet wide should be left in it, 20 or 30 feet above the lock, for the disposal of stray drift, and to permit a current through the upper entrance during rises, which would otherwise deposit sediment there.

The lower entrance is sometimes provided both with a land crib and with a river crib. The former acts partly as a protection to the bank, and has also to support the sediment which floods deposit behind it. The river crib, however, is of doubtful utility, although in some cases it has been built with the object of checking the effect of reactions from the dam on the lock gates. We are acquainted with locks, with and without river cribs, subject to similar conditions of flow, and in most cases the cribs could have been dispensed with. It is usually best to omit them until their advisability has become apparent.

On locks having movable dams there is rarely any necessity for a crib or wall either above or below the river wall, but guide walls next the shore should always be provided for convenience in locking tows.

Where cribs have to be sunk through water to their foundations, as is usually the case where piles are not used, the same methods are employed as described for sinking crib dams, except that the base cribs should not be over 20 or 30 feet long, unless the water is shallow. It is more important to secure a thorough bedding than with a dam, since any settlement will cause the crib to lean. Where the foundation has been dredged out it is a good plan to raise the end of each base crib a foot or two, with the dredge or other power, and then let it fall, continuing the "shaking" until the timbers appear to have found a solid bed. The top can then be leveled up with shims or blocks at the water-line.

If concrete is to be used for a foundation, it is usually necessary to build a coffer-dam.

The cost of timber guide-cribs with stone filling, including all labor and material needed to complete them ready for use, varies from two to three dollars per cubic yard of total contents, and from forty to fifty-five dollars per thousand feet B. M., on the basis of the timber.

Accessories.—For holding craft while locking, and to prevent the currents from the valves from bumping them against the walls, check- or snubbing-posts on the coping, or line-hooks built in recesses in the faces of the chamber walls, are required. The former may be made of wood, or, better still, of cast iron, and should be round and not less than 8 inches in diameter, and about 14 inches high. The latter are usually made of $1\frac{1}{4}$-inch round iron, forged to suit. Where the lifts are small line-hooks are very satisfactory, but for high lifts snubbing-posts are preferable, as the mooring-lines can be more easily tended.

Posts should also be provided on the guide cribs or walls, so that craft can be tied to them while waiting their turn to lock.

On some locks cast-iron chocks are provided at the ends of the chamber, set close to the edge of the land wall. The mooring-lines are led through these to snubbing-

posts, and the chocks prevent the ropes from being chafed along the coping as the boat rises or falls in the lock pit.

The ladders in the chamber should be of iron, and one should be set in the land wall just above the lower gate recess, and one in the river wall just below the lower gate, so the deck-hands can climb up or down as the boat enters or leaves the lock. In addition to these, one should be placed at the upper end in the river wall, just below the gates. Where the chambers are very long intermediate ladders should be provided, and if the dam is a movable one, a ladder should be placed on the outside of the river wall, thirty or forty feet above the crest.

Gauges, etc.—Gauges showing the depths of water on the miter-sills should be placed in the river wall, one at each end, where they can be conveniently read from the land wall. They may be cut in the masonry and painted, but the best material for them is tile, with the divisions and figures burned in black on a ground of white enamel. This kind is easily cleaned, and is unaffected by the action of the water.

On some locks ornamental tile panels have been built into the walls with pleasing effect. Two are used, each about 3 or 4 feet square, and one in each wall, one showing the number, etc., of the lock and the year of its completion, and the other displaying the official crest of the United States, or that of the Engineer Corps.

Finish of Walls.—The inside edges of the coping of the land and river walls should be rounded off to a radius of about 2 inches, to prevent chafing of mooring-lines. It is good practice, in fact, to round off all coping edges, as they are liable otherwise to become chipped and disfigured. The tops of the walls should be crowned about one half-inch for drainage.

CONCRETE LOCKS.

General Design.—The use of concrete for locks was practically untried until 1892, although it had been used for other hydraulic works for many years. So far only two objections have been made to it, one, that its appearance is inferior to that of a lock of cut stone, the other, that it may not prove durable. The latter objection can only be answered by experience, but there seems no reason why concrete should not last as well at least as the softer classes of stone, since the latter are considerably affected by water and by weather. Another objection sometimes urged is that a concrete wall, being divided into sections, is less strong than a wall where each stone is bonded, but where the foundations have been properly designed this objection has no appreciable effect in practice.

Concrete is generally used where its cost would be less than that of cut stone. The materials are usually obtainable near the site, and are easily handled, and require no skilled labor in placing. The last factor is frequently one of importance, since masons and stone-cutters are often hard to obtain, and when obtained cannot always be depended on. A concrete wall can also be built much more rapidly than one of stone.

The design for a lock of this class should provide outlines as simple as possible, and offsets, curved surfaces, and difficult intersections, which are rarely necessary, either in the walls or in the culverts, should be avoided. This is desirable as it lessens the expense of the forms, and also permits them to be set up more rapidly. A good deal of delay and extra work may be caused by having to stop concreting in order to change or set up forms for changes of surface. Similarly the number of bolts, castings, or other parts requiring to be set during construction should be reduced to a minimum, as it is more difficult (contrary to the usual belief) to place them properly in a concrete wall than in a wall of stone.

Square corners on exposed surfaces should be avoided, as they are easily chipped off.

The walls should be divided into sections not much over 30 feet in length, otherwise they will crack open with shrinkage in drying and with temperature. Where long walls have been built in one length such cracks have appeared at intervals of 25 to 50 feet, and even where sections 45 feet long have been used a slight crack has appeared close to the center. While such partings are no more weakening than artificial joints, they are far more unsightly. These joints, when exposed to a considerable head of water, usually leak for some months after construction, as will also the main walls; but the action of the water on the carbonates in the cement, and the finer sediment of the river, will gradually seal them up. The leakage through the joints can be reduced by using mortar between the old and new surfaces as the wall is built.

Where the natural foundation can be easily drained, so that trouble from leakage or from caving material can be overcome without difficulty, the main outlines of the walls may be designed to start from the bottom, without offsets. Where, however, this condition is absent, as is usually the case with a rock foundation, it is an excellent plan to provide a footing course a few feet high, and a foot wider all around than the main body of the walls. Its top will thus provide a platform above the leakage, and the forms can be set up without fear of displacement from caving excavation, and with the leisure necessary for accurately lining them in.

Proportions and Materials.—The proportions of the mixture in the earlier locks were considerably richer than those in later ones. On the Illinois and Mississippi Canal (1894) they were one part of Portland cement to seven or eight parts of sand and gravel or broken stone. On Locks Numbers 1 and 2, Big Sandy River, Ky., and W. Va. (1902), they were one part of Portland cement, three parts of sand, and six parts of mixed gravel or mixed broken stone; while on Lock No. 9, Kentucky River, Ky. (1902), they were one barrel of Portland cement, 15 cubic feet of common river sand, and $33\frac{1}{3}$ cubic feet of mixed broken stone, from $\frac{1}{2}''$ to $2\frac{1}{2}''$ in diameter, giving a mixture of about 1 to 12. The last mixture possessed when hardened an abundance of strength.

At Lock No. 2, on the Mississippi, near St. Paul (1900), sand cement was used instead of pure cement, the proportions for grinding being one part of cement to one

View Showing the Construction of a Concrete Abutment.

(To face p. 169)

part of sand. One part of the resulting product was then mixed with approximately three parts of sand and six parts of stone, the proportions thus being 1 to 19 on the basis of the cement alone. The grinding plant was owned and operated by the United States.

All the above proportions were by measure, except that in some cases the cement was taken as packed, in others as loose measure. Specifications should always state whether the cement is to be measured loose or packed. The contents of a barrel when shipped vary with different brands from 3.03 cubic feet to 3.35 cubic feet, and when loose from 3.75 cubic feet to 4.19 cubic feet, according to the fineness of the grinding. Tests made from a number of different brands gave an average of 3.18 cubic feet in the barrel, and 4.07 cubic feet loose, the ratio thus being 1 to 1.28.

Sometimes a natural cement is used for the parts below water, for the sake of economy, but Portland cement should always be used above.

The sand should of course be clean, and, where obtainable, of coarse grains, and a good average quality will be obtained by having it fill the following specifications for fineness: To pass a No. 30 sieve, not over 70 per cent; to pass a No. 50 sieve, not over 50 per cent; to pass a No. 100 sieve, not over 2 per cent.*

The stone may be either broken stone or gravel, and should be free from dirt. For the former limestone is best, as it appears in course of time to combine chemically with the cement, but any good hard stone will answer. If the stone is soft, it is liable to crush under the rammers, besides being deficient in tensile strength. Certain classes of sandstone, however (especially those impregnated with iron), which are soft when quarried, will become reasonably hard after some weeks' exposure to the weather, and can then be used, where better stone is not obtainable, with fairly good results.

Our experience has led us to conclude that a denser and more uniform concrete is obtained where the stone or gravel does not exceed 1 inch or 1½ inches in diameter, and includes all smaller pieces down to ¹ of an inch in diameter. Where stone of 2 or 2½ inches in diameter is used it bunches together and leaves voids, which with the smaller stone are much less frequent, since it packs closer under the rammers.

It is occasionally impracticable to secure sand or stone that can be made entirely clean, and in such cases a small amount of earth or foreign matter need not be a drawback, as it will have no appreciable effect in the mass of concrete. Some specifications limit this amount to 2 per cent. † Similarly the gravel will usually retain more or less sand after screening, the variation in the amount being usually limited to 6 per cent.

---

* In certain localities it has not been possible to secure a coarse sand without great expense, and in such cases common river sand has been used, and has given results amply strong for all practical purposes.

† Experiments by the authors with some briquettes composed of one part of Portland cement to one part of sand and river mud, and with others composed of one part of Portland cement to three parts of sand and river mud, showed that where the mud did not exceed five per cent of the amount of sand no appreciable diminution of strength occurred, and it was not until the amount exceeded ten per cent that any marked reduction was observed.

The stone should be thoroughly wetted before being put in the mixer, but the sand should be dry, or not more than moist, as when it is wet it is more difficult to incorporate it thoroughly with the cement, and the varying amount of water in it will cause a constant variation in the wetness or dryness of the concrete.

Of slow-setting and quick-setting cements we believe the former to be preferable, as with the latter, if delays occur, the concrete may be spoiled, besides which, the feet of the men ramming keep the surface in the forms more or less disturbed, and if it has begun to set before the next layer has covered it, the bond will be destroyed. A cement which will not take an initial set in less than an hour, at a temperature of 70° Fahrenheit, is well suited for this class of work.

The use of different qualities of cement, or different proportions of the same cement, in the same section of a wall, is not a satisfactory practice from the point of view of construction. It has been done sometimes to save expense, a Portland cement being used for the outside 2 feet or so of thickness, and a natural cement for the interior of the walls, or using a richer and poorer mixture respectively of the same cement. In the former case it requires a constant attention from the cement house till the concrete is in place, in order to see that the proper cement is put into the mixer when called for, and in either case careful watch must be kept to see that when mixed it is placed in its special zone in the wall, and even with the best of care the batches will sometimes be misplaced, leading to delay. It is claimed by some, moreover, that as Portland and natural cements usually have different speeds of setting, the bond between them is not as strong as is desirable. In certain walls built with such combined concrete a parting between the two kinds has actually occurred.

The practice of embedding stones in the walls for the purpose of saving cement is rarely an economical one, unless they can be set without causing delays in the concreting.

Forms.*—The forms, or timber work inside which the concrete is placed, and which give the shape to the finished work, vary considerably in design. They are of two general types, one where planks are laid horizontally, held apart at the proper distances by tie-rods and struts, and without outside posts or supports, and the other where posts are used to support these planks, or lagging, as they are technically called, well braced to the ground or to other supports, and without any tie-rods, except perhaps at the top.

In the former type the plank are placed one or two at a time, and raised up when the concrete has hardened, and placed ready for the next layer above. After a section is finished the ends of the rods are cut off and covered with cement, or small turnbuckles or gas-pipe sleeves are used, allowing the ends to be unscrewed and withdrawn, the turnbuckles remaining in the wall. This type, although effecting a considerable economy in the cost of the forms, has not met with much favor, because it

---

* See also specifications for a lock—"Forms," Appendix B

is very difficult to keep the plank in line and to surface during ramming, the appearance of the wall suffering thereby, and because the concrete can only be built in small layers, instead of in large blocks or as a monolith.

With outside posts this difficulty is greatly reduced, and better results obtained in other ways. Much lighter timbers are used now for this class of form than in the first locks, where the posts were 8 inches by 10 inches, 4 feet apart, and the lagging 4 inches thick. For the same purpose posts are now used 3 inches by 8 inches, 5 feet apart, with lagging 2 inches thick, and the results are equally satisfactory. Where the lagging is of $1\frac{3}{4}$-inch by 12-inch plank, the posts should not be more than $4\frac{1}{2}$ feet apart, and if it is of $1\frac{3}{4}$-inch by 8-inch plank, they should not be more than 4 feet apart.

The principal object to be secured, where it is desired to obtain a good appearance, is thorough stiffness in the posts and braces. The old idea that the concrete brought against these a liquid pressure has been proved by experience to be erroneous, and the real force which distorts the timbers comes from the rammers, and to withstand it a great deal of stiffness is required.

All lagging plank, where a good surface is desired, should be seasoned and dressed on all sides, and the edges of the posts should also be dressed. The additional cost is small, and unless it is done bad joints and gaps will be visible everywhere. For the same reason old lagging, unless well cleaned and retrimmed where needed, should not be used for the best class of finish. The plank may be from 6 to 12 inches wide, some engineers preferring a narrow width on the ground that many close joints are less noticeable than a few wide apart, and also that narrow plank do not warp as much as wide ones. Yellow pine appears to be more satisfactory for this use than other kinds of timber, as it is less affected by the moisture, and less liable to warp and shrink. Matched lumber has been tried once or twice with a view to lessening the marks of the joints, but with indifferent results.

The raggedness of the vertical and horizontal joints of the blocks, where the wall is built in sections, may be avoided by the use of triangular strips of wood, of cross-section like a 45-degree triangle, and about $\frac{3}{8}$ of an inch on the sides. One of these is placed at each corner of the block and the facing rammed around it as the block is built up, and others of the strips are laid along the top edges when they are finished for the time being. When the adjoining block is built up, or when the first block is continued, similar strips are placed against those in the facing, and the result shows, when the forms are removed, as V-shaped grooves of smooth outline. Care is of course needed in making and setting the strips, so as to have them all of one size and placed true, and the horizontal joints must all occur at similar levels.

In one or two cases the lagging has been lined with galvanized or plain sheet iron, well greased, and this is said to have given an excellent finish, doing away with the joint- and grain-prints which are inseparable from wooden lagging, and which cause, on the score of appearance, one of the principal objections urged against concrete locks.

It is probable that a combination of galvanized sheet iron and triangular strips

would produce a surface of excellent appearance. Thus, before the concrete was put in, the sheets would be coated with a thin grease and placed horizontally against the lagging, and the strips would be set with a level and a plumb-line to represent joints, with half-strips at the end of each block as before described. If 36-inch sheets were used (which might be of No. 20 gauge) in lengths of 6 feet, the horizontal strips would be placed about 36 inches apart, covering the longitudinal seams, and vertical strips would be fitted between them about 6 feet apart, staggered in each course and covering the vertical seams, thus giving the appearance of a jointed wall. A somewhat similar method, but without the use of iron, has been used for small concrete work, greatly to the advantage of its appearance. The lagging should be dressed as usual, to secure evenness of face.

This procedure would of course entail some expense, but it would be a very small percentage of the cost of the work, and the gain would probably more than justify the outlay. A certain amount of labor would be saved also, that of having to make good edge and butt joints in the lagging.

The forms for the culverts and other inside openings may be made of rough plank, 1 inch in thickness. With stiffening frames 2½ feet apart this size will be found strong enough.

The posts, which may be of 3-inch by 8-inch lumber, should be rigidly braced on the outside, as the ramming has a powerful effect in springing them. With posts of this size a brace should be placed every 7 feet or closer in the vertical plane. Sometimes the tops are tied together with pieces which carry at the same time a track for dump-cars, at other times no ties are used. Where the concrete is deposited by dump-boxes such ties are a good deal in the way.

The following sizes for braces will be found satisfactory. For spans up to 10 feet, use 2″ by 4″ lumber; for spans of 10 to 16 feet, use 3″ by 4″ lumber; for spans of 16 to 22 feet, use 3″ by 6″ lumber; for spans of 22 to 28 feet, use 4″ by 6″ lumber; for spans above 28 feet, use 4″ by 6″ lumber stiffened as required. Where long braces are placed almost horizontally, it it usually necessary to shore up their centers.

This bracing is one of the objections to this type of form, as it takes a large amount of timber. Possibly the eventual solution of the problem will be found in principles similar to those employed in the lock near St. Paul, Minn.,* where the long timbers were trussed with tie-rods instead of being supported by braces. It has also been suggested to use built-up posts of triangular outline, stiffened like the legs of a bridge traveler, thus avoiding the long braces, while another method, which has afforded successful results where the lock was built in blocks, consists in imbedding in the concrete, about a foot below the top of the block being finished, ¾-inch bolts, one to alternate posts, and provided with a nut and washer each end. When the block is to be continued, a 4″ × 12″ timber is passed over the bolts, the posts with dapped ends rested upon it, the bottom course of lagging put on, and the whole screwed tight

* Annual Report Chief of Engineers, U. S. A., 1900, Appendix BB.

against the masonry. The tops of the posts are fastened to a similar timber, and the latter is joined to the corresponding one on the opposite side by tie-rods 10 feet or more apart, so that the dump-buckets can pass between. These tie-rods, and consequently the tops of all the posts, are then held in rigid position by wire guys on each side, running to the bank or to the coffer-dam, and tightened by means of long turnbuckles or eye-bolts at their ends. When the forms are removed the bolts, which should be greased before being imbedded, are drawn out, and the holes closed with mortar. In one example the posts were of $3'' \times 8''$ timbers 4 feet apart, and it was found that for a vertical span of 10 feet between supports they were scarcely stiff enough to withstand the ramming. This could be remedied by providing intermediate tie-rods and sleeve-nuts. This method has been advantageously modified by using guys on one side only, combined with braces. The arrangement of the longitudinal timbers was similar to that before described (except that an intermediate course was used) those opposite being joined together by $\frac{5}{8}''$ tie-rods provided with gas-pipe sleeves about 2 feet from each end. The heads of alternate posts on one side of the wall, at distances of about 10 feet, were provided with eye-bolts to which wire guys were attached, and just below good braces were secured to the posts. The latter were then brought to place and held rigid by adjusting each guy and its brace at the same time. The opposite row of posts was then adjusted by the tie-rods, which passed through the horizontal timbers, light struts being used on the top ones to assist in holding them apart. The final adjustment of each post, where required, was secured by wedging behind it against the timbers.

If cast-iron quoins are to be used, the sections should be bolted in place on a long timber, which must be perfectly straight and true. This should be then set up in exact position and thoroughly braced so as to hold the quoin plumb and rigid during the concreting. Where the forms extend from bottom to top, the sections are sometimes bolted directly to the lagging.*

If the lock is to be built by contract it is preferable to have the design of the forms carefully worked out, and the drawings and specifications for them made a part of the agreement. Few contractors have had much experience in building concrete locks, and their knowledge of what is required for satisfactory forms is consequently limited; while, on the other hand, the collective experience of the Government engineers is large and should be sufficient to permit of obtaining the best results. If the design is left to the contractor he will naturally endeavor to use cheap methods and materials, whereas to secure a form that will give straight surfaces and a good finish to the walls a considerable expense must be incurred, and unless suitable designs are exhibited to the bidders as a part of the contract, friction and dissatisfaction will later result. Nothing is more annoying in this respect, either to engineer or to contractor, than to have during concreting a constant lining-in of forms and lagging which have got out of place because of want of stiffness in the posts or for lack of proper bracing.

---

* See paragraph on "Quoins," in this chapter, and on "Anchor Bars and Pintles," in the chapter following

In some cases it has taken the entire time of an inspector, besides the time of employees of the contractor, to watch and correct changes in this portion of the work. Once the form has been set up and brought to line it should need little more attention, and where proper experience has been brought to the design this result can be secured with but small extra expense.

Mixing and Placing.—In mixing the concrete, which should always be done by machinery, the materials are usually dumped into the mixing box all together, the water poured in on top, and the box revolved. In one type of mixer the water is introduced after the box has commenced to revolve, the shaft being made hollow and pierced with holes, and connected by a pipe with the water-tank. While this method is preferable in theory, it has drawbacks in practice, as the holes become clogged with mortar or spalls, resulting in an uneven mixture, and a necessity for cleaning them out every few hours. Fewer turns are required if the materials are first revolved dry and the water then introduced, but this is rarely done in practice, and equally good results are obtained by the method usually followed.

No special order need be observed in putting the materials into the box, except that the water should be put in last, nor is it necessary first to mix the sand and cement. From eight to sixteen turns will be required, depending on the size of the box, which should not be more than about one-half full. The amount of water will depend on whether "wet" or "dry" concrete is to be used, and also on the dryness and temperature of the air. By dry concrete is meant concrete which requires long and hard ramming before any moisture shows on the surface, while wet concrete shows it with much less ramming, and soon begins to quake or shake like jelly, showing that it is worked into a mass. If the water runs in trickles over the surface it is too wet.

While dry concrete, carefully mixed and placed, appears from samples to give a stronger result than wet concrete, engineers seem to be coming to the conclusion that such a result depends on conditions hardly realizable in practice. For a dry concrete the amount of water must be gauged almost to a quart, and as the total needed per charge will sometimes vary 25 per cent between 6 A.M. and 6 P.M. on a hot day, owing to the evaporation, it is very difficult to obtain the exact amount required. Moreover, when the concrete is exposed in the forms, evaporation from it is very rapid, and if the water dries out before the concrete has set, the latter is ruined. It is not practicable to keep it covered while the men are working, and if water is sprinkled over it the cement is liable to be washed in, exposing the sand and stone. Dry concrete can thus be easily spoiled, while the same causes will affect wet concrete to a very much less degree. The latter also packs more closely in ramming, and with less work, and judging by experiments made in the same wall, is less porous when subjected to water under pressure.

When the concrete is mixed it is taken to the forms by dump-cars, or in dump-boxes handled by derricks. Frequently a track is laid in the lock pit on which a traveling derrick runs, taking the boxes from stationary derricks or from cars, and moving

back and forth as required. Where cars are used altogether they are run on a track laid over the posts, and placed far enough to one side to allow dumping into the forms below. This method is objected to by some engineers on the ground that the dumping tends to separate the materials, but it has been used in several locks where the fall was from 25 to 35 feet, and no trouble of this nature was experienced. The spreading of the concrete with shovels, necessary for ramming, will assist in correcting any tendency to separation, should it become apparent.

After the concrete is in the forms it must be spread out in layers 6 to 8 inches deep and rammed. The ramming is done with cast-iron rammers, about 6 inches square on the face, and weighing with the handle from 18 to 30 lbs., according to the ideas of the engineer in charge, our own experience being that where laborers are kept at ramming all day considerably better results are obtained by using the lighter weights.

Valve-seats and other ironwork can be set as the masonry is built up, while bolts for ladder-supports, frames, etc., can be built in by boring holes in the lagging for them and letting them project to the desired amounts.

After a section is finished it should be covered with tarpaulins for about two days, or until well set, and be wetted twice a day for about two days more, if the weather is warm, as a large mass of concrete is very thirsty.

The forms can be removed in from three to five days, depending chiefly on the conditions of the temperature, and any rough joints or edges in the facing should then be rubbed down with a piece of wood or of soft stone. The facing is usually soft when the planks are taken off, but it will quickly harden by exposure to the air and sun.

Facing and Coping.—If facing is to be used—which should be done on all surfaces which will be exposed in the completed work—the templates for it should be put in just before the layer of concrete is spread. This facing should not be less than an inch thick at any point, and should be mixed with one part of cement to not more than two parts of sand, as it has to stand the weather and the abrasion of boats. It should be mixed with a very small proportion of water, just sufficient to make it stick together when squeezed hard in the hand, but not sufficient to show after being well rammed in place. The templates for it should be about 2 inches higher than the thickness of the layer of loose concrete, and wider at the top than at the bottom, so they can be pulled up after the concrete is rammed without disturbing it. A convenient size may be made by taking a 2-inch plank, and planing one edge down to an inch in thickness, providing staples or rings on the other edge for pulling up. The plank should be in lengths not exceeding 6 feet, with shorter ones for corners, etc. When a layer is finished these templates are pulled out, and the facing put in and rammed in layers (which should not be over 3 or 4 inches deep) with a special rammer. This may be formed of a piece of bar-iron, 1 inch square and 4 inches long, provided with a bent handle.

The facing may be mixed by hand, or in the mixer, dry, and wetted just before being put in place. This mixing must be thoroughly done, and the sand used should

be fine rather than coarse. If made in the mixer, the box must be first cleaned from any stone or gravel that may have remained in it.

The insides of the culverts need not be faced, as the plain concrete is sufficiently strong to withstand the abrasion of the water.

The coping is usually finished off by taking some of the facing mortar, mixed a little more wet than for the walls, and spreading it on the surface of the concrete before the latter has begun to set. This mortar is then floated over with a straight-edge and brought to a level finish. This method secures a good bond between the surfaces, but the top lacks the smooth appearance to be seen on good cement sidewalks, and which it is very desirable to secure. To obtain it, the work should be put in the hands of an experienced sidewalk mason, or if such is not available, an ordinarily intelligent mason, acting under instructions, can produce fair results. The mortar is spread as before mentioned, and just before it attains the final set it is carefully troweled over, causing the cement next the top to rise to the surface and giving a smooth finish. Division lines, if such are used, are cut with a special tool, something like a plasterer's float, but having a curved V-shaped iron on its under side, with which the lines are struck. The work should be carefully protected from the sun until hardened.

If the workman is inexperienced, a few sample blocks should be made and finished off before he is allowed to attempt the main walls.

If the coping finish were put on after the concrete had set, the operation would be much simplified, but it may be doubted if the bond would be lasting. We are acquainted with cases, however, where new copings have been put on walks and have lasted without sign of failure. In such cases the old surfaces are well roughened and washed over with a grout of neat cement before the finishing coat is put on.

As the coping of the different blocks is usually not ready for finishing at the same time, and as it is often not possible to secure a competent mason at any time his services may be required, the blocks can be built up to within an inch or two of the finished height and be left low in the center, so as to provide a good base of fresh concrete on which to lay the finish without having to leave or put up forms again. Thus if the edges of the wall are to be rounded to a 2-inch radius, the facing or concrete may be built up and carefully leveled off at 2 inches below the coping. From this level the concrete is made to slope down at about 45 degrees to a depth of a foot or more, and then carried flat to meet the opposite slope, leaving a trough in which a body of fresh concrete can be placed whenever the coping is to be put on. By this method the coping when commenced can be finished without interruption, the troughs being filled up as required.

Monolithic Concrete.—In some locks the concrete has been built in courses a few feet in height, placed at different times; in others it has been built in one block, of the full height of the wall, and from 20 to 30 feet long. The former method, as shown where concrete work has been removed, does not possess the strength of the latter, and requires a constant moving of men and appliances from one point to an-

other. The latter method, which was supposed to require that work on the block must be continuous, has been modified in recent practice, and results equally good for all practical purposes have been obtained by working in daytime only. With this system the surface of the masonry is left in a ridge or a hollow along the center, and carefully leveled at the edges and struck off with a trowel at the end of the day's work, so there will be no ragged joints or wire edges. On resuming work next day the top of the concrete is wetted, and cement scattered over it and brushed in, thus forming a thin grout. The joints show very slightly in the finished work. If a joint of appreciable thickness is employed, the mortar should not extend to the face, unless it is made of facing material. By this means night-work can be avoided, an arrangement which is always desirable, as it is very difficult to inspect concrete properly at night, and in addition forms sometimes become displaced or rammed out of line without being noticed. With monoliths it is also easier to secure uniformity of outline, since the forms are set up at one time for the full height of the block.

Each section of the wall should be bonded into the adjacent ones, not only for stability but also because the tendency to leakage through the joint is thus reduced. In the first locks this bonding was made by placing vertically one, and sometimes two timbers, about 8 inches wide and 4 inches thick, along each end of the section being built. When the forms were removed the timbers were taken out, leaving grooves into which the concrete of the next section bonded. We have observed, however, that any settlement of the walls always breaks off these tongues, and a better method is to take V-shaped troughs, which may be made of 1″ × 12″ lumber, and set them on end, either close together or with a foot or more between, the end ones being kept a foot or two from the face of the wall. These will provide a large number of bonds without any weakening of the masonry, and will hold the sections together against unequal lateral movement.

Cost.—The cost of a lock, including coffer-dam, excavation, construction of walls, back-filling, valves, gates, and all labor and material needed to make it ready for operation, varies from ten dollars to fourteen dollars per cubic yard for masonry of cut stone, and from nine dollars to twelve or thirteen dollars per cubic yard for masonry of concrete. These figures are obtained from examples of locks built within recent years, the wide variation resulting from differences in cost of material, accessibility of site, and other causes incident to all engineering works.

CHAPTER III

LOCK GATES AND VALVES.

GATES.

Miter-gates.—The gates used for closing each end of a lock usually consist of a pair of symmetrical leaves, movable about a vertical axis, shutting against each other at one end and against the miter-sills at the bottom, and abutting against the hollow quoins at the other end. That part fitting into the hollow quoin is called the heel, while the opposite end is called the toe, and in old construction, where a post formed each side of the gate-frame, they were called respectively the heel- or quoin-post and the toe-post. The gates rest on shoes which turn upon pivots or pintles at the bottom of the heel portions, and they are held vertical by means of collars or anchor-bars fastened to the top of the masonry, and passing around pins in the bonnets or tops of the gate-heels. A roller has sometimes been placed on the bottom of the gate near the toe, traveling on the floor and relieving the strain on the anchors. The gates are usually operated by means of spars or chains, or both, connecting with the top of the gate near the toe and with suitable capstans located on the walls.

The recesses for mitering gates should always be at least one foot longer than the gates when these are open, and should afford a clearance of about a foot behind them to allow space for floating sticks and débris, which would otherwise hinder them from opening fully.

The lap of the gate on the miter-sill may be made from 4 to 6 inches, and the clearance between the bottom beam and the floor may be 6 inches or more.

Single Swinging Gates may be seen on many of the older locks of narrow width, but they are rarely applied to the locks used in river improvement. They are practically a single leaf of the miter-gate. Usually they have a balance-beam extending back over the wall and serving as a lever for their movement. At the toe they are supported against a shoulder in the wall.

Rolling Gates are in use in Europe and on the upper Ohio River in America. In the locks on this stream, where the chambers are 110 feet in width, the gates are supported on a heavy track, and present an appearance not unlike a long box-car. When not in use they are rolled back into recesses in the bank. There are, of course, two gates, one at each end of the lock. Each is about 14 feet high, and about the same

width, and has a length of 118 feet. They are built of steel, trussed like a bridge. The movements are made with heavy chains wound upon drums by steam-power, and considerable difficulty has been encountered in some cases with the breaking of chains, and also with the breaking of the axles upon which the gates rest. The recess, which is 120 feet long, reaching back into the river-bank, is also a source of considerable expense in construction, and the accumulation of débris or of ice in it is an annoyance at the time of locking. These difficulties are occasionally increased by the breaking of parts under water, so that this style of gate, while of ingenious design, is still of uncertain utility.

Tumble-gates.—A gate with horizontal axis, known as a "tumble"-gate, forms a type which has long been in use on the Erie Canal and in Europe. These gates consist of a single leaf hinged to the lock floor, the boats passing over it when it is lowered. They are maneuvered by chains passing over crabs on the walls.

Hydraulic Gates.—It has been suggested to apply the principles of bear-trap and drum dams to lock gates, in order to secure a gate which could be operated automatically, and several ingenious devices have been proposed for such an application. Gates of the former type are in use at the lock of the Sandy Lake Dam, in Minnesota, and are 40 feet in length with a rise of 13 feet. They were built in 1895, and serve as sluice-gates also. An upper gate of the Chittenden drum type is in existence at Lock No. 2, on the Mississippi River, near St. Paul, having been put in about 1901. These are believed to be the only locks where hydraulic gates are in use.

Automatic gates which could be raised and lowered easily would possess several advantages over mitering gates, such as permitting the washing out of deposit from the chamber and entrances, etc., but they would be more difficult to keep in proper working order, and more expensive to repair, owing to the greater number of parts which would be always under water, and it may be doubted whether they will have any extended application.

Calculations for Lock Gates.—*General.*—In the following calculations only those types of gates will be examined which fall within the ordinary practice of the engineer. The rarer types, such as arched gates, have been discussed in other works.*

The simplest form of gate is that of a beam placed squarely across the lock chamber, and supported against the masonry at each end. This style has been used on canal locks, as before described. The strains in such gates are those of a simple beam, or, where the gate is large enough to require truss framing, they can be analyzed graphically or by moments, in the methods employed for framed structures supporting uniform loads.

* See "Mitering Lock Gates," First Lieutenant Harry F. Hodges, Government Printing Office, Washington, 1892. An analysis of the arched gates of the St. Mary's Falls Canal is published in the Annual Report of the Chief of Engineers, U. S. A., 1895, Appendix LL; and a full list of literature on Lock Gates will be found in the Report of the Deep Waterways Commission, Washington, 1900, pp. 200–207.

The next form of gate, and the one which is practically universally used, consists of two leaves, and is known as the mitering type.

Gates of this type are subject to the following strains:

(1) The pressure of the water.

(2) The reactions from the miter, the quoins, and the miter-sill.

(3) The weight of the leaf.

(4) The upward pressure from the water under the bottom beam. Shocks from craft, twisting from drift caught between the miters or behind the heel, will also induce strains, but they cannot be calculated. Experience has shown, however, that a gate designed to meet ordinary strains and properly connected, is very rarely disarranged by drift, and will stand in addition a considerable blow from a boat.

The pressure (4) seldom needs consideration, as it is usually offset by the weight of the framing, and where it is not, a proportion of the friction against the quoin may be taken into account. This upward pressure is a maximum when the gate has its maximum load, and at that time the friction is also a maximum. If it is not desired to make any allowance for its effect weights must be added to the gates, or other means taken to counterbalance the upward pressure.

The weight (3) in wooden gates is supported by wooden struts or by diagonal ties of iron, as will be mentioned later on.

The strains from (1) and (2) are usually those which determine the proportions of the framing. The reaction from the miter-sill affects only vertically framed gates, except in so far as it supports the lowest beam of a horizontal framing.

In determining the head to be provided for in calculating gates for river locks one of the most important points to be considered is that in stationary dams there is usually a difference of level between the pools until, and sometimes after, the water has risen to the top of the walls. We are acquainted with one lock where there is a difference of 9 feet between the pools when the lock is "drowned out," or flooded by the river. This fact will, of course, require the upper part of the gate to be made strong enough to stand the head produced by the difference between the pools at the varying stages of the water, a difference which, in locks already built, can generally be determined from gauge records. In new locks it can only be approximated from a study of general conditions, or by comparison with other locks similarly placed. On canal locks and those on rivers with movable dams these variations do not occur.

The lower gates, both with fixed and movable dams, should be proportioned so that they will withstand a full pool above, and a reduced pool below, since the latter condition will occur sooner or later through the leakage in fixed dams caused by wear, and the possibility of repairs to the movable dams. In certain cases the upper gates may have to be used as a coffer-dam, and it should be seen that they will stand the accompanying head of water without danger.

Water Pressure.—*Horizontal Framing.*—Let P (Fig. 8) be the pressure per lineal foot on any beam K of a horizontally framed lock gate AB, H the head of water to the center of K, and w its height in feet. If the beam supports sheathing, w must of course include the total depth of water pressure supported by the beam, and if there be water below, H will be the net head.

Then P equals

$$w \times 1' \times H \times 62\tfrac{1}{2} \text{ lbs.} = wH \times 62\tfrac{1}{2} \text{ lbs.}$$

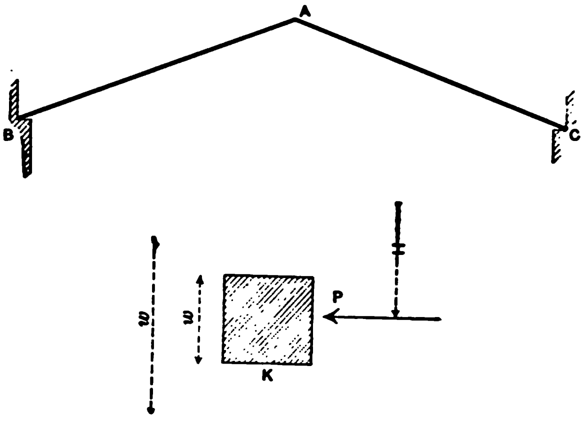


FIG. 8.

per lineal foot. The bending moment M, if there are no concentrated loads from valves, etc., will be

$$\frac{P \times (AB)^2}{8} \times 12 \text{ inch-pounds} = \frac{3\,wH \times 62\tfrac{1}{2} \text{ lbs.} \times (AB)^2}{2}.$$

The section required for this bending can then be found by the usual formulas $M = \frac{sI}{c}$ and $I = \frac{wt^3}{12}$, where s = the extreme fiber stress per square inch, c = distance from neutral axis to extreme fiber, t = thickness of beam, I = the moment of inertia of section, and M and w are as just stated.

Where K has to support concentrated loads from valves, we have a combination of distributed and concentrated loads, which may be calculated as follows:

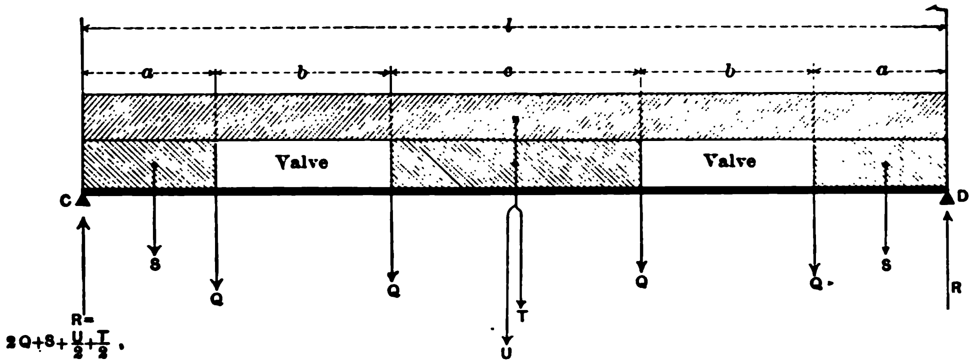


FIG. 9.

Let CD (Figs. 9 and 10) be a beam supporting two valves which produce loads on CD at their points of support, each equal to Q. The beam has then to carry the valve loads, the proportion of water pressure on the lengths a and c between the valves, and the proportion of uniform load from the water pressure in the panel ECD above.

The proportion of the pressure on a between CD and F which is carried to CD is

$$a \times \left\{\frac{e\,(H+d)}{2} + e\cdot\frac{e}{2}\cdot\frac{1}{3}\right\} \times 62\tfrac{1}{2} \text{ lbs.} = \frac{ae}{6}\left\{3\,(H+d)+e\right\} \times 62\tfrac{1}{2} \text{ lbs.} = S.$$

Similarly the proportion of pressure on c is

$$c \times \left\{\frac{e\,(H+d)}{2} + \frac{e^2}{6}\right\} \times 62\tfrac{1}{2} \text{ lbs.} = \frac{ce}{6}\left\{3\,(H+d)+e\right\} \times 62\tfrac{1}{2} \text{ lbs.} = T.$$

The proportion of the uniform load between E and CD, extending the length of the beam, which is carried to CD is

$$\left\{\frac{ld\,H}{2} + ld\cdot\frac{d}{2}\cdot\frac{2}{3}\right\} \times 62\tfrac{1}{2} \text{ lbs.} = ld\left\{\frac{H}{2} + \frac{d}{3}\right\} \times 62\tfrac{1}{2} \text{ lbs.} = U.$$

The reaction at C is therefore $2Q + S + \frac{U}{2} + \frac{T}{2} = R.$

The bending moment at the center is then

$$\left\{R\cdot\frac{l}{2} - Q\,(b+c) - S\left(\frac{a+c}{2}+b\right) - \frac{U}{2}\cdot\frac{l}{4}\right\} \times 12 \text{ inch-pounds.}$$

From this we can find as before the section required. As the center of the beam is the point of maximum moment, it should be seen that it is not too much reduced by notching out for the gate-straps which are usually placed there for the purposes of construction.

Where more than two valves are used the process of analysis is similar to that given above.

In a metal gate where cover-plates are used it will of course be necessary to find the moment at one or two other points to determine the length of plate required.

The calculations for the upper beams of a gate are the same as used for beams with uniformly distributed loads as given in the first example.

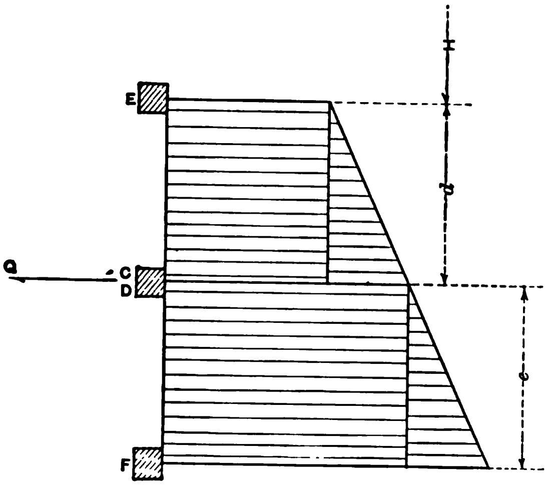


FIG. 10.

In the foregoing calculations we have not taken into account the support afforded to the beam through its being connected with those above and below. Such support exists, but its amount is uncertain, as it depends on theoretical perfection of jointing which may or may not exist, and which in any case is liable to change as the gate wears. It is best therefore to proportion each beam so that it will carry its load independently.

It is frequently necessary in wooden gates where valves are used to reinforce the valve beams so they will carry the total load with safety. This is done by framing a metal girder between them, with diaphragm or vertical plates to which the valve journals are attached; the metal and the beam combined thus furnish the strength required.

The bottom beam of a gate which rests against the miter-sill can be considered as partly or wholly supported by it, and will therefore require no unusual section.

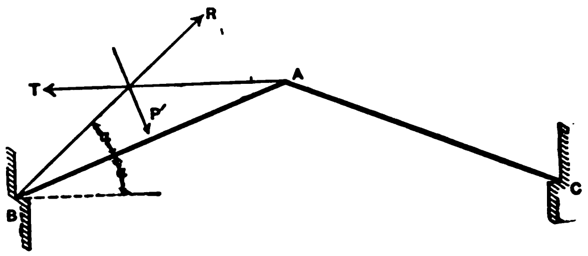


FIG. 11.

End Reactions.—In addition to the water pressure on its face, each beam is subject to end pressure from the opposite gate and from the hollow quoin. Thus if P' (Fig. 11) be the total water pressure on a beam inclined at an angle α, and T the reaction at the toe, we have, by moments about B,

$$P' \times \frac{AB}{2} = T \times AB \sin \alpha, \quad \text{or} \quad T = \frac{P'}{2 \sin \alpha}.$$

The reaction R at the quoin will be found to be of the same value as T, and inclined to AB at the same angle.

The component of T which acts along and produces compression in AB is equal to

$$T \sec \alpha = \frac{P'}{2 \sin \alpha} \times \sec \alpha = \frac{P'}{2} \cdot \cot \alpha.$$

This compression is distributed over the section of the beam, and produces a stress per square inch equal to $\frac{P'}{2} \cdot \cot \alpha \div a$, where a = the area of section in square inches. On the upper or compression side of the beam this stress increases that due to the water load; on the lower or tension side it decreases it.

It will usually be found that the section required for bending will be sufficient to carry this additional compression without undue strain, since the point of maximum stress occurs only at the edges of the middle of the beam; the material near the axis of the beam thus receives very little strain from bending, and in all ordinary cases it will be found sufficient to carry the load from the end reactions as well.

Vertical Framing.—Where the beams are placed vertically, as in a certain class of metal gates, they are supported at the bottom by the miter-sill and at the top by a horizontal girder. The strains in the latter are those from the concentrated loads of the verticals, while the strains in the beams themselves are found as follows:

Let H (Fig. 12) be the head of water, and d the distance from the top support to the

surface, w the width of water supported by the beam, R_1 and R_2 the reactions as shown. Then the pressure $P = wH \cdot \frac{H}{2} \cdot 62\frac{1}{2}$ lbs. $= \frac{wH^2}{2} \times 62\frac{1}{2}$ lbs.

Taking moments about the base we find

$$R_1 = \frac{PH}{3\,(H + d)} = \frac{wH^2 \times 62\frac{1}{2} \text{ lbs.}}{6\,(H + d)}.$$

The bending moment at any point distant x from the surface is

$$\left\{R_1\,(d + x) - wx \cdot \frac{x}{2} \cdot 62\tfrac{1}{2} \cdot \frac{x}{3}\right\} \times 12 = w \times 62\tfrac{1}{2} \cdot \frac{H^2\,(d + x) - x^3\,(H + d)}{6\,(H + d)} \times 12 \text{ in.-lbs.}$$

The section required may be determined by the formulas given on page 177.

Strain on Anchor-bars, Diagonals, etc.—The anchor-bars should always be proportioned to carry the weight of the leaf when swinging in air, and in the case of small gates an increase should be provided in the section found necessary as a precaution

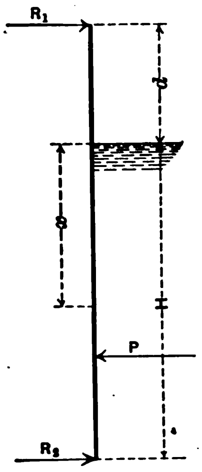


FIG. 12.

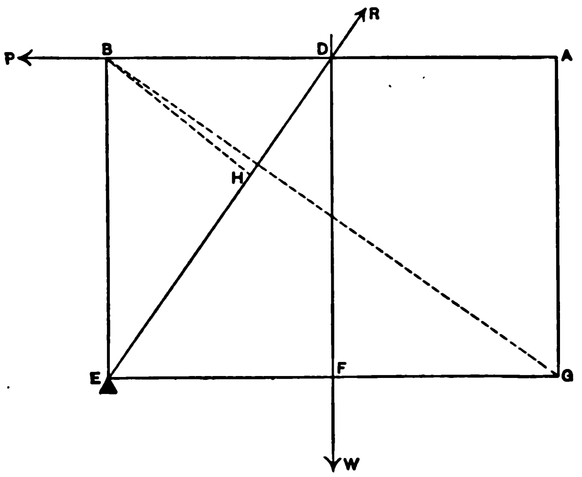


FIG. 13.

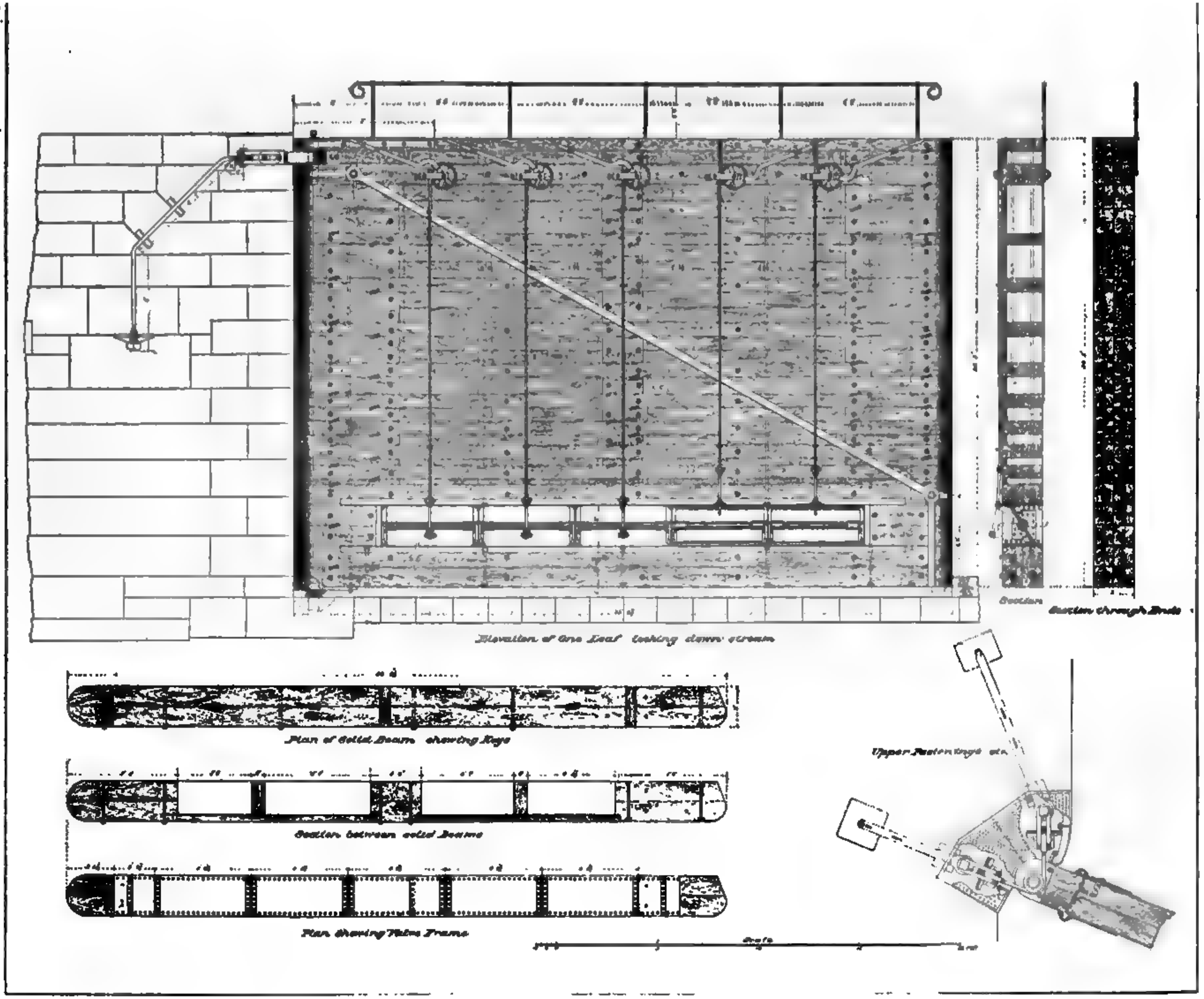
against blows from craft. The use of rollers to support the toe of the gate, with the object of relieving the pull on the anchors, has been discarded, since any deposit over the track interferes with the wheels, and when they are a little worn their support becomes practically valueless.

If W (Fig. 13) represents the weight of a leaf swinging free, the pull on the anchor-bars, taking moments about E, is $P = \frac{W \times FE}{BE}$.

The thrust R on the pintle E, taking moments about B, is $R = \frac{W \times BD}{BH}$.

The strain on the diagonal BG is $\frac{W}{2} \times \frac{BG}{AG}$.

With a diagonal on each side of the gate the load on each will of course be one-half of this.



GENERAL DRAWING OF A LOCK GATE AS USED ON THE KANAWHA RIVER, W. VA.

(To face p. 180.)

Unit Stresses.—*Wood.*—In a wooden gate it will generally be found that only tensile stress need be considered, as if the pieces are made strong enough in that respect the section will be ample for all stresses of compression and of shear. Those parts of a gate which usually sustain a full head, such as the beams above the lower pool surface, may be designed for the minimum unit stresses given below, while those parts which are rarely subject to maximum loading, such as the beams of the lower gate below the lower pool, which can support the greatest head only when the lower pool is drawn down, may be designed for the maximum unit stresses given.

A safe working tensile stress for gates of white pine is 900 lbs. per square inch, with a maximum of 1200 lbs. In a few cases the first unit stress has been made as high as 1200 lbs., but where the timber was subject to constant exposure the strains induced were found to be somewhat high. For oak and yellow pine a tensile stress of 1200 lbs. per square inch may be used, with a maximum of 1600 lbs.

Steel.—The unit stresses used for steel gates have varied considerably. Thus on the gates of the St. Mary's Falls Canal, Mich., (1895) 9000 lbs. per square inch was used for the general framing, with a maximum of 10,000 lbs. One authority recommends 10,000 lbs. per square inch for compression and 12,000 lbs. per square inch for tension, while in several examples of gates for locks in rivers a maximum of 12,000 lbs. per square inch has been used for compression and 16,000 lbs. for tension, the beams or girders in these cases being assumed to receive no additional strength from the sheathing plates. On the gates for the proposed locks of the Deep Waterways from the American Lakes to the Atlantic (1900), the flange stress of the main girders was limited to 10,000 lbs. per square inch, part of the sheathing being assumed to act as a flange-plate. The pressure between the wooden quoin-posts and the masonry was limited to 400 lbs. per square inch.

As a gate is rarely subject to greater loading than that of static water pressure, it would appear safe to use moderately high unit stresses, making allowance as far as practicable for deterioration by rust

WOODEN GATES.

Vertical vs. Horizontal Framing.—Vertically framed wooden gates are practically obsolete in America, and for all river locks of ordinary size the simplest and best method of construction consists of horizontal beams extending in one length from the toe to the heel, well bolted and strapped together, their ends being shaped to fit the miter and the hollow quoin respectively. This gives a solid timber from end to end, and avoids the weakness of beams jointed into vertical heel- and toe-posts, such as are found in the older styles of gates.

Spacing of Beams.—In wooden gates, except where they are very small, the upper portion of the gate is usually paneled, that is, the beams are placed at varying distances, and the spaces between are closed with 2-inch or 3-inch plank. This is an

excellent method of construction, as it saves material and weight, and also reduces the buoyancy of the gate in high water. The beams are usually made of the same width and depth as those below, and placed at distances apart which will make each one support the same unit stress.

Sizes of Beams.—The following are the horizontal widths of beams used in actual practice in certain horizontally framed wooden gates:

For lock chambers 27 feet wide, and heads up to 10 feet, the beams are 12 inches wide.

For lock chambers 36 feet wide, and heads up to 16 feet, the beams are 15 inches wide.

For lock chambers 52 feet wide, and heads up to 18 feet, the beams are 18 inches wide.

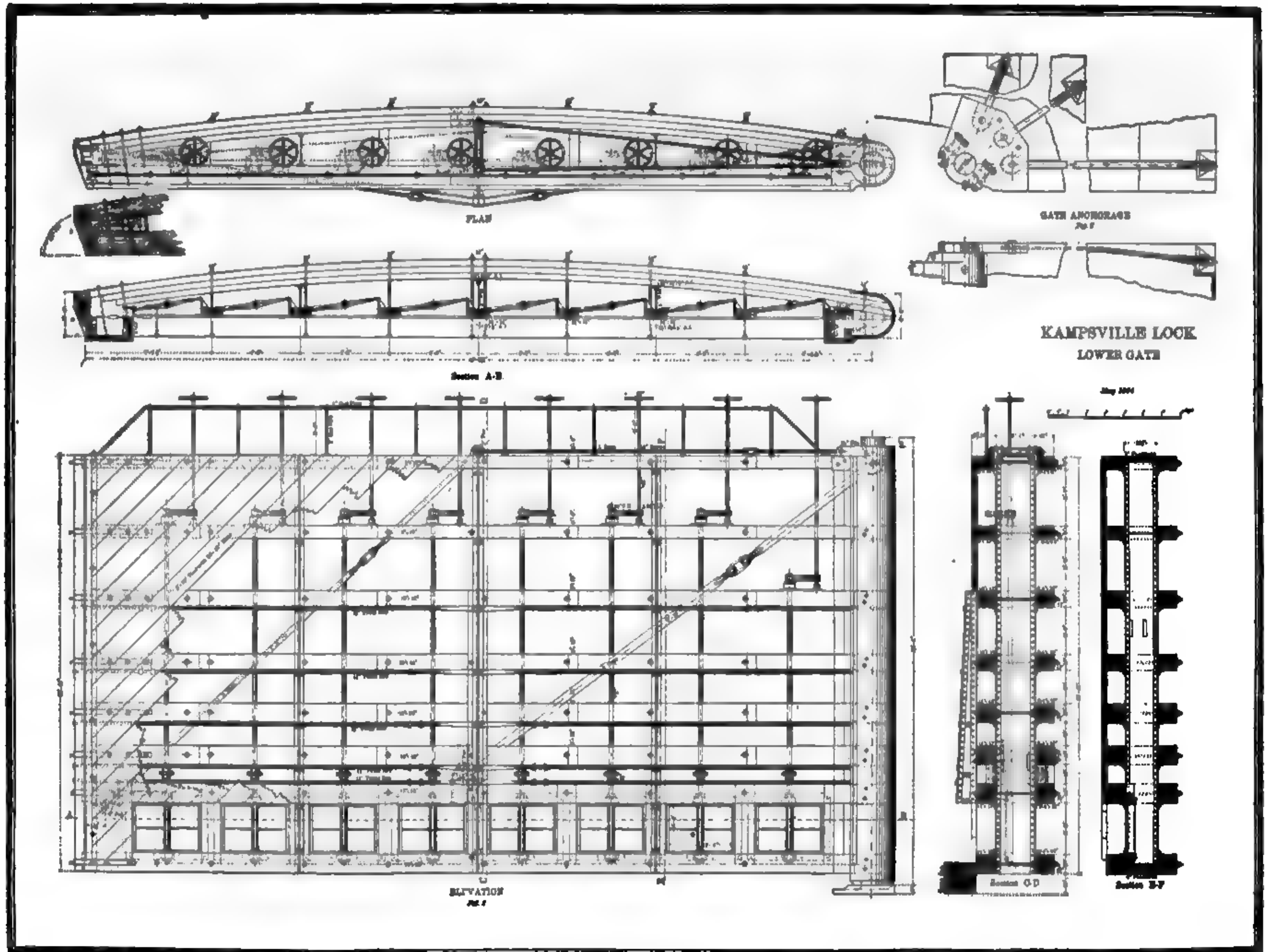
In some cases two beams, placed side by side and keyed together, have been used instead of one single large beam; thus we find examples where two beams 9 inches wide, and other examples where two beams 12 inches wide, have been used instead of one single beam 18 inches wide. This practice, however, is not a satisfactory one, as it takes more labor in construction and depends for proper strength on the exact fitting and duration of the keys. The cost of two small timbers is of course less than that of one large one, but the additional labor required in framing and handling the former will more than offset this. Experience with double- and single-timbered gates on the same river has shown, moreover, that the former are more easily injured by shocks and accidents.

Kind of Timber.—In most gates white oak has been the timber used, and where sound sticks can be obtained, and especially if they have grown on bottom lands, there is no better material. Yellow pine has been largely used of late years, as oak is becoming scarce; it has proved very suitable, and costs a little less in the framing than oak. White and Norway pine are also good, but they are not often used except in Northern streams on account of the expense. Hemlock is not suitable, as it rots easily.

On the Manchester Ship Canal, in England, Demerara greenheart was used in preference to metal in all the gates. This is an unusually durable timber, although it may be questioned whether this fact would compensate for the advantages to be gained in large gates by the use of steel.

Assembling.—Where gates are built of horizontal beams of the full length these are held together by 1¼ or 1½-inch bolts, running vertically through their centers, from the top beam to the bottom one, or by metal straps placed on the outside, and let in flush with the faces of the gate. Where the latter are used those at the heel and toe should be provided with turnbuckles for use in cramping the timbers together during construction. In each case diagonal straps must be used to hold up the gate; these are preferably formed of eyebars provided with turnbuckles, one end passing over a pin in the bonnet, the other over a similar pin near the lower end of the toe.

If the timber is to be dressed at the site, the pieces should be ordered ¼ inch larger than the finished size, to allow for planing. With oak or other timber liable to



GATE OF THE KAMPSVILLE LOCK, ILLINOIS RIVER, ILL.

(To face p. 182.)

warp it is best to allow more than this. Sometimes the pieces are planed to exact size at the mill, but this plan has its drawbacks, as the timber usually shrinks and becomes more or less scarred in transit and in handling, and if it gets in wind there is no spare material for trimming it off. By using it ready planed some expense is saved, but the work is rarely as satisfactory as can be obtained by dressing at the site.

The beams should be finished so that when the gates are closed there will be no space between their ends, or between them and the miter-sills. If they are too long they will always leak, as the ends do not appreciably wear in use, and if too short, recourse must be had to filler-blocks, which are always liable to be knocked off. When the gates are built up in place it is best to leave the toes rough, and to saw them off to the exact length and bevel when the gates are finally swung, taking the measurement along each sill. These measurements should always be recorded on the drawings, as they usually vary slightly, so that when the gates have to be replaced the new ones can be cut to an exact fit.

In new locks the gates are usually framed on shore and then built up in place piece by piece, but in replacing gates in old locks they must be set up complete, so as to delay navigation as little as possible. This is effected by building them on shore on ways, and launching them when finished into the river. They are then supported under a barge and taken into the chamber, when they are hoisted in place by crabs on the wall or by other means.

METAL GATES.

In the last few years gates of metal have been coming into favor in the United States, and several examples of this class of construction are to be found on different rivers, while many others are proposed. They will doubtless be more widely applied, since timber suitable for gates is becoming more expensive every year, and the quality of that which can be obtained is steadily deteriorating. From a comparison of several cases we find that at the present time metal gates cost from 30 to 60 per cent more than wooden gates, depending on location and on size, the larger gates being relatively the cheaper. As to duration, the consensus of opinion seems that the former class will last about twice as long as the latter. It should be remembered, however, that where the metal is constantly submerged it can never be properly cleaned and painted. The effect of water on steel, which is the only metal obtainable in this country from which plates and shapes are rolled, appears much more destructive than upon cast or wrought iron, and a few years' submersion will corrode and pit the metal to a very serious extent, rendering it a practical impossibility to secure a thorough cleaning. It might be possible to pump out the lock and thoroughly clean the gate every year, but this would involve a stoppage of navigation, and even on rivers of small traffic the loss and expense would scarcely justify the gain. In addition to this there would always be some recesses or portions of the construction, as for

instance the under side of the bottom beam, which could only be cleaned and repainted with great difficulty, if at all.* For these reasons the parts constantly under water should possess a large excess of strength, and all the metal in them should be at least $\frac{3}{8}$ inch in thickness. The lower part of a wooden gate will usually outlast two upper parts, but of steel gates it is probable that the reverse will be found to be true.

On the continent of Europe steel has replaced wood almost entirely, even in very small gates, but in England wood is still largely employed.

Vertical and Horizontal Framing.—Metal gates for locks of ordinary size may be divided into two classes, those with vertical framing, and those with horizontal framing. In the former class the beams supporting the water stand vertically, and are supported by the miter-sill at the bottom and by a horizontal girder at the top. In the latter class the beams are horizontal, as in a wooden gate, and are framed into vertical heel- and toe-posts, which in turn transmit the strains to the walls and to the opposite leaf.

There is also a mixed type in which vertical girders are employed to assist in carrying the loads from the plating to the horizontals, the latter forming the main supports. With this construction both horizontals and verticals act in resisting the pressure, and the precise loading of each can only be approximated by a tedious mathematical investigation. For this reason many designers assume that all the pressure is resisted by the horizontals, and make each vertical strong enough merely to carry the pressure to them from its own panel. This type is also more complicated in construction than a plain system of horizontal or of vertical framing, and as a rule is therefore not economical.

Where a gate is very shallow in proportion to its length, economy of metal favors the use of vertically framed gates. Some engineers, however, claim that the saving is comparatively small, even in extreme cases, and does not offset the disadvantages. These are, that the top horizontal girder brings a heavy strain high up in the wall, and that the recess in the coping necessary for it when the gate is open, reduces the width of surface required by the lock-tenders for the proper operation of lock, or else necessitates a wider wall in order to avoid such a reduction. To these objections, it might be added that when the lower part of a vertically framed gate is rusted away the entire leaf must be renewed. With horizontal framing the gate could be cut in two at the beam nearest the water, and a new bottom riveted on, the upper portion being still retained in service.

Vertically framed gates are only applicable to new locks, on account of the girder recess above mentioned.

Design.—The economical design of a steel gate requires few members with wide spans between, thus concentrating the strength and allowing a good thickness of

* One of the most effective coatings for preserving ironwork under water is the ordinary red-lead and oil paint, while pitch has also given excellent results in some localities. In certain of the French seaport locks the metal is galvanized, which process is said to add about ten per cent to the cost.

metal throughout. This will necessitate the use of buckled plates, which, though more expenisve than flat plates, are much cheaper in the end, as they will carry a much heavier load. They should be $\frac{3}{8}$ inch thick at the bottom, or heavier if obtainable, diminishing to $\frac{1}{4}$ inch for the upper panels. All shop-work should be made as simple as possible, since curving or bending in order to save metal frequently costs more than the material it saves.

The calculations for the beams are similar to those before given, and careful attention must be used in securing proper connections between the various parts. The diagonal suspenders used on wooden gates to uphold the toe are unnecessary on metal gates, as the skin-plates supply all the stiffness required to prevent sagging. Their omission is also preferable for navigation, as craft entering or leaving a lock occasionally rub along the gates, and the down-stream diagonal is liable to be bent by the impact.

Where the gates support considerable pressure it is sometimes necessary to plane off the ends of the lower beams so that they will have a close bearing against the heel- and toe-verticals, and thus transmit the compression from the other leaf. Stiffeners may also be used for this purpose.

The most economical shape for the beams for locks up to 55 feet in width, with ordinary lifts, consists of I beams, which can now be obtained in depths of 24″ and under. They are cheaper than built-up girders, as they require less shop-work, and where the section has to be reinforced, cover-plates can be added to the flanges. Holes should be provided in the webs for the drainage of rain- and flood-water.

The heel- and toe-posts are usually provided with timbers bolted to them and shaped to fit the hollow quoins and the miter. This is an excellent practice, as the wood provides a tight and elastic cushion, and where the quoins are of masonry it will wear itself into fitting any irregularities, and where they are of cast iron it can be planed off to fit imperfect joints and alignment.

The wooden heel is objectionable, however, on the point of difficulty of renewal, and care must be taken in designing the gate to see that this is reduced to a minimum. Where this is done timber will generally be found preferable to a metal heel cushion, owing to the difficulty of adjusting the latter to the unavoidable imperfections of construction.

It has been proposed where a metal cushion is employed to use strips of a soft metal, such as lead, set vertically in slots in the cast-iron pieces. The lead would then permit trimming to fit inequalities of the quoin.*

Where wooden cushions are used, they should be cut in two just below the pool level, and the portions above, which will rot first, can then be removed without disturbing the parts below water.

The contact with the miter-sill may be formed by the bottom beam of the gate, or an additional wooden cushion may be used.

* Proceedings American Society of Civil Engineers, April, 1902.

Sheathing Plates.—These may be of buckled or of flat plates. The former are preferable, as they possess much more strength than the flat plates, and permit of wider panels between the beams without requiring vertical stiffeners. Their actual strength is a matter of experiment, as it cannot be calculated. The following values were found for plates 3 feet square, well bolted down on all sides.*

Thickness.	Safe Load per Sq. Foot (One-fourth Breaking Load).
$\frac{1}{4}''$	1120 lbs.
$\frac{5}{16}''$	1540 "
$\frac{3}{8}''$	2240 "

Where the plates are supported on four sides, but not bolted or riveted down, the above loads are to be reduced one-half. If two sides only are supported the loads are to be reduced in the proportion of 8 to 5.

The resistance of flat plates under conditions such as exist when they are employed as skin plates is likewise a matter of uncertainty. The most recent and probably the most reliable experiments in this field are those of Professor Bach of Stuttgart, who gives the following formula for flat rectangular plates, subject to fluid pressure:

$$t = cb\sqrt{\frac{p}{\left\{1 + \left(\frac{b}{a}\right)^2\right\}f}},$$

in which t = the thickness of the plate in inches;

c = a constant = 0.61 for a plate riveted down all around;

a = length of plate between supports in inches;

b = breadth of " " " " "

p = intensity of fluid pressure in pounds per square inch;

f = maximum allowable tensile stress in the metal in pounds per square inch.

The spacing of the rivets in the plates, as regards securing water-tightness of the joints, need not be closer than 6 inches, and examples are in use where the spacings were made 9 inches, and have proved perfectly tight under heads of 14 to 16 feet.

Single and Double Sheathing.—The use of double sheathing for gates, that is, of plates on both up-stream and down-stream sides, has been chiefly confined to very wide locks. The object of the double plating is to provide water-tight compartments near the bottom of the leaf so as to secure buoyancy and consequent reduction of weight. This lessens the strain on the anchors, and facilitates maneuvers. In practice, however, they have hardly proved an unqualified success. It has been found difficult to keep the chambers free from leakage, and when the water gets in the gate loses its buoyancy, and the second skin merely acts as an additional and useless load, defeating the very purpose for which it was put on. This style of gate, where

* Pocket Companion Carnegie Steel Co., 1900.

used on rivers, has been found open to another objection, which is, that the upper or non-water-tight compartments fill up with mud. This can only be removed through the manholes, and with much trouble, and if left in, adds considerably to the weight of the gate and causes it to rust.

It would appear that single-sheathed gates, although less rigid than those with double sheathing, are preferable for all cases which are likely to occur in the ordinary practice of the engineer. For the few cases which may require extraordinary dimensions, a study of the local conditions must of course decide the choice of type.

It may be added that both for single and for double sheathing the anchorages and maneuvering apparatus should be proportioned to take the heaviest strain that can result from the construction, that is, when the gate is swinging in air.

Combination Gates.—In order to avoid the loss of strength resulting from the rusting of iron and steel under the water, a style of gate has been proposed in which the parts constantly submerged would be of timber, while those above water would be of metal. We are not aware of any case where this idea has been put into practice, but there appears to be no reason why it could not be successfully applied.

DETAILS OF CONSTRUCTION, ETC.

Anchor-bars, Pintles, and other Fittings.—The simplest style of anchor-bar for locks of all but unusual size consists of an ordinary eye-bar provided with a turnbuckle, one eye being connected with the gudgeon or bonnet pin, and the other being placed over a large bolt cemented in the masonry, and provided with a collar or casting at the top to distribute the pressure. The bars are placed horizontally in recesses which are covered with cast-iron plates, and are thus accessible at all times for examination or adjustment. They require a minimum of forging, which is a desirable feature, not only because of economy, but also because this class of work is not always reliable. Not more than one bolt should be used to each bar, as where two or more are used it is not possible to set them to an exact enough position to secure an equal strain on each, and as the result only one of them actually bears the load.

Usually two anchor-bars are used for each gate, one placed in continuation of the line of the gate when closed, the other placed a little back from the same line when the gate is open, so as to avoid bringing the bolts too close to the edge of the masonry. This gives the gate a slight tendency to over-balance when open, which is usually counteracted by the second anchor-bar. Should the motion exist, however, it will be so slight as to be of no practical importance.

The strain on anchor-bars, except for very large gates, should not exceed 5000 lbs. per square inch, as they occasionally have to withstand the impact of boats, etc.

Two forms of pintle are in use, one in which a steel pin forming the support is set into a cast bed-plate, the latter being set into and flush with the masonry; the other, in which the support and the bed-plate are cast in one piece and set about an inch

into the masonry. The former supposes that the pin can be easily removed and replaced when worn, which will occur sooner or later; but in point of experience it is usually very difficult to do this, as the surfaces, even when greased, adhere so firmly that steel wedges sometimes fail to separate them. As the bed-plate is cemented in, the removal may finally necessitate coffering the lock, since it is difficult to do such work under water. With pintles of other styles, which have proved equally and sometimes more durable in practice, this objection does not exist, as they can be replaced without difficulty.

The surfaces of contact of a cast pintle and a cast shoe should always be chilled, and if this is done they will wear longer than pintles of steel or shoes provided with special bushings.

Pintles for very small gates are usually about 4 inches in diameter at the top, and for locks of 55 feet in width, about $5\frac{1}{2}$ or 6 inches in diameter.

In concrete locks the pintles may be placed directly on the masonry, using a unit-pressure of 200 lbs. to the square inch, or special bearing-stones may be employed. The former method is considerably cheaper, and is not less successful than the latter.

The bonnet, or connection at the top of the heel, is sometimes formed in wooden gates by using two iron straps bent to fit the heel, one at the top of the gate, the other about a foot lower, with a small casting for the gudgeon-pin to which the anchor-bars are connected. A more usual method is to employ a casting which fits over the top of the heel, as shown on the drawings. It should always be provided with a movable cap, so the anchor-bars can be removed and replaced when required during repairs to the gate. In some of the older designs, where this was not done, it is necessary to raise the entire gate when repairs have to be made which require a removal of the bars.

The gudgeon-pins vary from about 2 to 6 inches in diameter, according to the size of the gate.

The center of rotation of the gate should be placed a little up-stream of the center line, so the leaf will swing free of the quoin as soon as it begins to move. This distance may be from $\frac{1}{2}$ inch to several inches, according to circumstances.

All the pins should have $\frac{1}{32}$ to $\frac{1}{16}$ of an inch clearance in the pin-holes so that they can be removed easily.

The diagonal straps used with wooden gates are sometimes made in one piece, and sometimes in two, joined by a turnbuckle. In the former case the straps are made a little short, from $\frac{1}{4}''$ to $\frac{3}{8}''$, and are put in place by being heated and shrunk on, and in the latter they are put on as ordinary straps and screwed tight. This is the preferable method, as should one become bent or broken by a boat or from other causes, it is little trouble to remove and replace it. No strap or other iron should be placed within 2 inches of the vertical surfaces of contact of the toes.

The rollers on which the gate-spars travel (which may be two in number for each spar, one on each edge of the wall) should be of ample diameter, to minimize the friction. Light and serviceable ones can be formed by taking a piece of $3\frac{1}{2}$- or 4-inch gas-

pipe and closing the ends with cast plugs, in which are inserted short lengths of 1-inch rod to serve as journals. The journal-boxes should be arranged so that the rollers can be removed when desired.

Each gate should be provided with a hand-railing, placed on the upper side so as to be out of the way of boats. It may be made of $1\frac{1}{4}$-inch gas-pipe, with uprights of the same size, fitting loosely into cast-iron sockets bolted to the gate. On the approach of floods the railing can then be lifted out in one piece and removed.

Miter Joint.—The upper side of the miter or toe, both in wooden and in metal gates, should be beveled off for about a third of its width, so that the actual surface of contact when mitered will fall upon the lower two-thirds. This is done in order to throw the compression from the end reactions into the down-stream sides of the beams, thus relieving the tension existing in them from the direct loading, and at the same time avoiding increasing the compression which exists from the same cause in their up-stream sides.

Alignment.—The usual practice is so to place the gate in its recess that its face will be from 6 to 12 inches back from the face of the chamber when the gate is open. This is done to prevent boats from striking it, but unless fenders are provided extending out to the chamber line, boats will strike and break off the edges of the square and of the hollow quoins. These fenders should always be placed horizontally, not vertically.

In other cases the gate itself is made flush with the chamber and acts as the fender. Either way is satisfactory in practice, since craft rarely cause any injury to gates when they glance against them, if the diagonal straps and other projections have been provided with the proper fenders.

Bumping-blocks should be provided on the up-stream side of the toe of each gate, one near the top, the other near the bottom. Their object is to hold the gate parallel with the recess when open, and they must therefore be of a width such that when the gate is in this position the blocks will touch the masonry.

Gates for Concrete Locks.—The heels of gates for concrete locks should be designed so that a certain amount of trimming can be secured without difficulty, as in this class of masonry it is not practicable to secure as accurate a hollow quoin surface as can be obtained in a lock of cut stone.

Life and Care of Gates.—The life of a wooden gate in this country is from ten to twenty years, the latter being a limit which is very rarely reached. However, as the submerged part of a gate is not as subject to deterioration as the upper part, the leaf is frequently repaired by rebuilding the part above water, and retaining the submerged portion, which will usually be ready for renewal when this second top has become decayed. On the North German canals the life of a wooden gate is given as from 17 to 38 years, with an average of 25 years, the same average being given for the wooden gates at certain French harbor locks. An example is to be found of a wooden gate at Antwerp still in good condition after 40 years of service, the timber being creosoted fir, while gates of Demerara greenheart at some of the English locks have lasted for more

than 40 years, and some of those in the docks at Liverpool, built of oak, are said to have had 70 years of service. The comparatively short life of American gates is doubtless due to severer climatic conditions.

The life of iron gates used in canals in Germany has been from 28 to 48 years, with an average of 40 years, and in Holland similar gates have lasted for 60 years. Galvanized iron gates used in sea locks in France are estimated to last from 35 to 40 years.* Major Hodges, in his work on Lock Gates before alluded to, after quoting examples of iron gates constructed between 1850 and 1870, and still in use in 1892, gives the average duration of a metal leaf as forty years, when properly cared for and repaired.

The greater durability of wooden gates in Europe, as compared with those in America, is a factor largely in favor of their use, and metal gates, where exposed to salt water, do not appear to last much longer than those of wood. For such positions greenheart timber is frequently used, as it withstands the ravages of the teredo better than any other kind, although it is said to be subject to their attacks in tropical waters.

As the lock gates are so vital a point in a system of navigation, it is unwise to allow them to deteriorate until they become weak. We are acquainted with a case, the river in question being at the time under the control of a private company, where a set of lock gates were allowed to go without repairs until a portion of one of the lower leaves gave way. A raft was passing out of the lock at the time, and was caught in the resulting current, and before it could be checked struck the lower gates. The timbers were so decayed that the shock broke several more of them, increasing the current till in a few moments all the four leaves were carried out, and the river was pouring through the chamber. As it was the time of low water, the damage was not as serious as might have been anticipated, but it was many weeks before navigation could be resumed.

With fixed dams of high lift it is often best to open the lower gates just before a flood drowns the lock, and to fasten them securely in that position to the walls. This may cause some deposit of sediment in the chamber, but if the gates are left closed, the reaction from the dam may force them open and injure them seriously by hammering against each other and against the walls. This has happened in several cases. In one of them it had been the practice for several years to close the gates and tie them together with a 1¼-inch iron rod at the top. During one rise some unusual combination of forces occurred which tore the rod in two, thus setting loose the gates and injuring them greatly.

While this procedure will not always ward off damage, since examples are known where lower gates, fastened open, have been torn down by extremely violent rises, it appears to be a safer practice than that of leaving the gates closed.

Care must always be taken to see that wooden gates are not too buoyant when the water rises over them or they will float up at the toe and be liable to injure the top masonry of the quoin, and possibly displace the pintle. Cast-iron weights of preferably about 30 lbs. each are usually employed to counteract this tendency, being placed on the

* Papers read before the International Congress of Navigation, Düsseldorf, 1902.

arms of the gate at the approach of the high-water season, and removed when it has passed.

Operation of Gates.—For opening or closing gates the most common method in use, and probably the best one for ordinary conditions, consists in a spar, provided with a cast-iron rack, and worked by a gear-wheel on a capstan, or a spar without any rack and worked by two capstans and lines, one for pulling the gate open, the other for pulling on the outer end of the spar and closing it. The spars as a rule should be attached close to the ends of the gates, so as to utilize all the leverage available, although in many cases the attachment is made at a distance from the toe equal to one-third of the length of the gate, for the sake of using a shorter spar. Where the latter is long, and overhangs the wall, as on the river side, a light movable bracket carrying a roller is used to support the free end, attached to the wall in a way to permit its removal for an approaching flood.

In large gates chains are frequently used, attached to the bottom of the leaf, the closing chain crossing the chamber and passing up the opposite wall. Steam or water supplies the operating power. In several large locks of recent construction in Europe, as on the North Sea and Baltic Canal, the method of chains has been discarded, and a steel strut substituted, placed above water, where it can be easily reached for repairs, an advantage which chains do not possess. It is worked in some cases by an hydraulic cylinder, and in others by geared turbines, and would appear preferable in many ways to the use of chains.

The old method of a balance-beam has also been applied occasionally, the top beam of the gate being extended back to form an arm to which a rope is attached, wound by a capstan, or a link chain worked by sprocket wheels and levers.

Electric motors have been applied in recent years to the operation of gates, apparently with good success, and examples of them may be seen on the Canadian lock at Sault Ste. Marie, and on the Amsterdam Ship Canal.

Where practicable it is a good plan to place the spars below the coping, so they will not be in the way of the lock-tenders during maneuvers. This method is in use on the Moldau in Bohemia, and on the Fox River in Wisconsin, and has proved very satisfactory. The spars work in recesses covered by cast-iron plates.

No simple and practical formula has yet been given to determine the power needed to move a lock gate. With small gates the displacement of the water takes a large proportion of the power required, but with gates of moderate or large size the friction, and sometimes the effect of wind, are the chief factors to be overcome. We give below the sizes of spur-wheels, etc., used in certain examples, all the gates being operated by spars and pinion-wheels, turned by levers. The opening or closing under ordinary conditions could be easily accomplished by one man. The spars were attached near the ends of the gates, and the levers were 4 to 5 feet long.

Gate 12 in. thick, 15 ft. 9 in. long, 22 ft. 0 in. high, pitch diameter of spur-wheel 11½ in.
" 15 " " 21 " 9 " " 29 " 6 " " " " " " " 8 19/32 "
" 18 " " 30 " 9 " " 34 " 0 " " " " " " " 8 19/32 "

All the gates were of timber, the two last being paneled. Where the gates are of metal they require less power, being usually much lighter than corresponding sizes in timber.

At the Canadian lock at Sault Ste. Marie, above referred to, the electric motor used to operate a leaf about 37 feet by 40 feet is of about 25 horse-power, and at the lock at the same locality on the American side a hydraulic motor of approximately the same horse-power is used for a metal leaf about 43 feet by 50 feet. At the Cascade locks of the Columbia River, Oregon, a metal leaf somewhat similar in size was found to require about 10 horse-power for its operation under ordinary conditions, and at the large lock at Bougival on the Seine, near Paris, the hydraulic motor (consisting of a cylinder press attached directly to the gate) for operating a wooden leaf about 39 feet by 26 feet can exercise a direct force of 14,000 pounds.* The amount of power required will of course depend largely on the method and point of application.

The spars and operating gear for river locks should, where exposed, be all designed so that they can be detached without difficulty and carried out of danger from high water. Where this has not been done, damage from drift has frequently resulted.

Gates for Large Locks.—The question of the design of gates for locks of unusual size was taken up in 1899 by a Commission of Engineers, for the United States Government, in connection with the surveys for a proposed ship canal between the inland lakes and the Atlantic.† The widths of the proposed locks were 60 and 80 feet, and the lifts from 30 to 41 feet, with depths on the sills from 21 to 30 feet. This required some of the gates to be designed for possible heads of over 70 feet, far in excess of any yet employed. The investigations therefore were exhaustive, and the results are the more valuable as they combine international opinions and practice.

The most economical rise of sill was found to be about one-fifth of the chamber width, or about twenty-one degrees. The ordinary mitering type was adopted for the gates, as it possesses more than any other the merits of reliability, simplicity, and strength, and steel was selected for the material, wood being inapplicable to the conditions. The pure arch form, which has hitherto been principally used for wide-span locks, was discarded in favor of a girder section with a straight down-stream face and an up-stream face straight for about one-third of its length, and then curved in toward both ends. The middle was made 4 to 4½ feet wide, and the ends about 2 feet wide. Comparative estimates showed that an arch would not save more than 10 per cent of weight, while it would possess the disadvantages of being much more expensive in manufacture, and of having less stiffness and strength to withstand shocks, since its section would be thinner than that of the girder gate. It would also

---

* Annual Reports, Chief of Engineers, U. S. A., etc.

† Report of the Board of Engineers on Deep Waterways between the Great Lakes and the Atlantic Tidewaters, Washington, D. C., 1900. Appendix on Lock Gates by Henry Goldmark and S. H. Woodard.

require a deeper recess in the walls. For these reasons the girder section was found to be more economical and suitable.*

All the plating was to be of single sheathing, flat plates from $\frac{3}{8}$ to $\frac{1}{2}$ inch thick being used. The framing throughout was of the horizontal type of equal spacing, only two vertical diaphragms being put in each leaf, and these were employed solely for the sake of stiffness. The quoin and toe posts were of wood, except for gates for locks 80 feet wide, with heads of over 30 feet, in which cases steel had to be used, as wood was unequal to the pressure.

In order to overcome the upward pressure under the gate, as was necessary in certain cases, the arrangement was adopted of using a narrow vertical plate for the bottom panel, stiffened with angles and attached to a main frame just above by cast brackets. When the gate was closing the main frame would pass over the sill and be stopped by shoulders on the brackets which bore against the sill. These shoulders were to be about a foot wide, up and down stream, the spaces between them being closed by a horizontal plate stiffened with angles. By this means the water would press upward only on the horizontal plate, instead of on the whole width of the gate. The sill was to be trimmed to fit this special construction.

Cost of Gates.—The cost of gates varies considerably, as will be seen by the following figures, which give the cost of several wooden gates built by the United States Government by hired labor.

1896. Two pairs of gates of 12″ timbers, for a lock 27 feet wide and of 9 feet lift.

Cost of timber (13,000 feet B. M.)	$325.00
" " cast iron (14,000 lbs.)	423.50
" " wrought iron (9,400 lbs.)	553.50
	$1302.00
Framing, placing, and miscellaneous	1204.00
(about $92.50 per M.)	
Total	$2506.00

About $193 per M., or $2.20 per square foot.

(This includes cost of operating spars, butterfly valves, etc.)

* The comparative ability of the arched and of the straight girder type to withstand shocks may be illustrated by supposing a boat to strike, with sufficient impact to force the leaf open, the lower side of a girder supporting a head of water. In an arched girder, which depends for its equilibrium under a load on a fixed direction of the resultant pressures at the toe, the framing is suddenly submitted to intermediate bending stresses for which it was not designed, and which, if the pressure is great, will distort it seriously. In a straight girder, which acts as a beam, the point of support is merely changed from the toe to a point a few feet distant (as the point of impact when a boat strikes a gate is almost invariably near the miter), and the supporting power of the girder remains practically unimpaired.

1899. Two pairs of gates, of 15″ timbers, for a lock 36 feet wide and of 14 feet lift

Cost of timber (29,000 feet B. M.)	$690.00
Cast and wrought iron	824.00
	$1514.00
Framing, placing, and miscellaneous	1406.00
(about $48.50 per M.)	
Total	$2920.00

About $101 per M., or $1.45 per square foot.

(No valves in gates. Cost of spars included.)

1897. Two pairs of gates, 18″ wide, of two 9″ timbers, keyed, for a lock 52 feet wide and of 15½ feet lift.

Cost of timber (57,600 feet B. M.)	$944.00
Cast and wrought iron	634.00
	$1578.00
Framing, placing, and miscellaneous	2315.00
(about $40.25 per M.)	
Total	$3893.00

About $67.60 per M., or $1.32 per square foot.

(These had no valves in the gates. Cost of spars included.)

The cost of wooden gates on the North German canals, for heights of 30 feet and widths of chamber of 42 feet, is stated to be about $3.60 per square foot, and for double-sheathed iron gates of the same size about $4.50 per square foot, or one-fifth more.

The cost of wooden gates at certain French harbor locks is given as $5.50 to $7.00 per square foot for widths of chamber of 42 feet to 52 feet, and the cost of iron gates for widths of chamber of 69 feet as $8.20 per square foot.

On the Manchester Ship Canal, with a depth of water of 40 feet and a width of chamber of 65 feet, the cost of greenheart gates was about $41,000 per pair, the estimate for iron gates of the same size having been about $28,000 per pair, or one-third less. Greenheart timber was used in preference to iron on account of its superior durability.*

The cost of several sets of steel gates, erected between 1898 and 1902 on certain tributaries of the Ohio River, ranged from $2.70 to $3.20 per square foot in place, the heights being from 27 feet to 33 feet, and the corresponding lengths from 30 feet to 20 feet for each leaf.

The contract price of the five pairs of double-sheathed arched gates for the lock of the St. Mary's Falls Canal, Mich. (1893), was $182,610. The steelwork of the main framing was 5.9 cents per lb.; the cast iron, 6.0 cents; the cast steel, 9½ cents; the steel

* Papers read before the International Congress of Navigation, Düsseldorf, 1902.

forgings, 27½ cents; and the bronze, 35 cents. Of the first-named item there were 2,405,000 lbs., and of the others a total of 169,300 lbs. The width of chamber is 100 feet, with 21 feet of water on the sills, and a lift varying from 18 to 20.8 feet, the largest gate being about 43 feet high, and the smallest about 25½ feet. The cost per square foot was approximately $13.*

On the German canals just alluded to it is the practice to use wooden gates for locks up to 42 feet in width, and iron gates for greater widths. In Holland the use of wood is limited as a rule to widths of chamber of 60 feet.

VALVES.

The regulation of water in the chamber is obtained through valves placed in the gates or in culverts built in the walls and floors for passing the water around or under the gates. Culverts for filling usually discharge through the lift-wall by means of several small lateral openings connected with a large culvert reaching across the lock. This division of the water into small streams emptying near the lock floor causes little disturbance to boats. In large locks they are frequently extended along the main walls, instead of being built in the miter-walls, and discharge through openings at right angles to the axis of the chamber. In the lock of the St. Mary's Falls Canal the water flows along culverts under the floor and discharges upward through two hundred and fourteen openings into the chamber.

The emptying culverts are usually built around the heels of the lower gates, entering from the gate recesses and discharging below the miter-wall into the tail bay. The river-wall culvert sometimes opens directly through the wall into the river below the dam.

The wickets, or valves controlling the movement of water in the lock, are maneuvered from the top of the lock wall by machinery connected with the valves through openings in the walls, or by a system of sheaves and chains on the faces of the walls.

Valves in culverts are very satisfactory, although more expensive to construct and to repair than valves in gates, which are equally satisfactory in operation. They should be used for filling in cases where the lift-wall is above the lower pool and where gate valves would consequently discharge into the air and splash over boats in the chamber. Sometimes, instead of being placed in culverts, they are put in the floor above the miter-sill, and open directly into culverts passing under the latter.

The entrances to all culverts should be provided with screens of bar iron, about 2½ inches by ⅜ inches, hinged on the down-stream side to open like a gate, and latched to the wall. They are needed to prevent débris becoming entangled in the wickets.

Manholes have been provided in many of the older locks, opening vertically through the wall. In the later ones, however, they have been omitted, as they have not been found of practical use.

* Annual Report Chief of Engineers, U. S. A., 1895.

Balanced or Butterfly Valve.—There are several types of valves in use, each of which has its advantages and drawbacks. One of the most satisfactory is the balanced or butterfly valve. When used in the gates they usually turn on a horizontal shaft, and are worked by a lever and rack wheel from the top arm. They are easily repaired and not expensive. Another type turns in a vertical plane and is worked by a rod which runs through the valve and to the top of the gate, where a simple cranked lever is used for operating. This valve is simpler in mechanism than the former, but as the leverage obtainable is small, the valve also has to be made small. The following sizes of gate valves with horizontal shafts have been taken from examples of successful practice.

For heads up to 8 feet, each valve was 4 feet 3 inches long by 2 feet 5 inches wide.

For a head of 16 feet, each valve was 2 feet 6 inches long by 2 feet 3 inches wide.

These dimensions are about the extreme limits of size which can be operated for the respective heads by one man, and sizes for intermediate heads can be determined accordingly.

The valves may be of cast iron or of $\frac{3}{8}$-inch steel plate, stiffened. The latter material is preferable, as it stands accidents better.

Where balanced valves are placed in culverts they are usually made a little higher than wide, and turn on vertical shafts which connect with the operating gear, set in recesses in the coping. They may be of cast iron, or of $\frac{1}{2}$-inch steel plate stiffened with angles or beams. Cast-iron valves of this class have not proved very satisfactory in use, several cases having occurred where the castings have broken, sometimes through a piece of drift jamming them, sometimes for no apparent cause except that of the flaws inseparable from this material. They have also to be made much heavier than a steel wicket.

The shaft for a wicket of about 25 square feet of area should be $3\frac{1}{2}$ or 4 inches in diameter. If a cast wicket is used, the shaft should be forged square for the connection, not hexagonal, as with the latter shape the edges sometimes wear off, and the grip is lost. It should be slightly smaller at the bottom of the wicket than at the top, or wedge-shaped, and should only bear for about a foot at each end. This will permit of its being withdrawn for repairs without much trouble.

The wicket itself is set in a rectangular cast-iron frame, which is shaped to suit the masonry and provided with ribs projecting about an inch, against which the edges of the wicket close. These ribs should be provided with wooden strips bolted on, as this permits of trimming to a closer fit than can be obtained with metal only. The arms of the wicket may be made equal on each side of the shaft, or the arm which opens inward may be made a little longer than the other, on the principle that as soon as the opening begins the water will act on this additional length and assist in the maneuver. While this principle is doubtless correct, it appears to be of little practical use, as the violent action of the currents in the culverts is so uncertain in different cases that we have seen unequal-armed wickets open automatically in one lock and close automati-

cally in another, the general design being similar. Under these conditions it is believed to be best to use equal arms and to provide gearing powerful enough to permit one man to control the opening or closing at any point without unusual effort.

In one example, on the Kentucky River, where the wicket was $4\frac{3}{4}$ feet wide and 6 feet high, with a head of $15\frac{1}{2}$ feet (measured from the bottom of the wicket), the pitch diameter of the large gear-wheels was 39 inches and of the pinions $7\frac{11}{16}$ inches. The ratio of the power at the end of the operating lever to the turning force delivered at the circumference of the wicket shaft was 1 to 600, friction being neglected. Under the full head one man could operate it, but with difficulty, but with the head reduced to about 13 feet the operation was easy. On other locks with similar conditions, but less powerful machinery, two or more men were needed for the maneuver, and where additional help was not available, serious injury sometimes resulted to the operator because of the wicket getting beyond his control. It is much better therefore to have an excess of power than to have a deficiency.

The fit of the gear-wheels on their arbors and on the shafts should not be a "machine-shop," but should be a little looser, or it will be very difficult to get them apart for repairs.

Gridiron Valve.—Another style of gate valve is the gridiron or sliding valve, in which the valve is pierced with rectangular openings. It is usually worked by a rack and worm, but it is an undesirable type, as it is slow in operation and small in area of discharge, although permitting but little leakage.

Drum Valve.—The drum valve, known in France as the Fontaines valve from the name of its inventor, consists of a cylindrical ring 4 to 6 feet in diameter, and about 18 inches high, sliding up and down in a second ring above, the latter having a closed and water-tight cover. The culvert, which has a circular orifice, opens under the ring, and the latter rests when closed upon the edges of this opening, cutting off the water. To permit the passage of the water the ring is pulled up by machinery on the wall and slides up in the second ring, thus uncovering the culvert. This is the easiest valve to operate, as the water-pressure balances itself on all sides, but it is costly and requires large openings in the masonry, so that the water can have a free approach all around. Examples of the type are to be found in America on the Muskingum, the Kentucky, and the Big Sandy rivers.

This type can also be arranged with little extra expense so that the water-pressure will operate it, and some ingenious patents have been devised to accomplish this end.

The height to which the valve should lift should be not less than one-quarter of the diameter of the culvert, in order to secure the full capacity of the latter.

An improvement on the ordinary type of drum valve would be to put the closed cylinder on the inside and the open sliding cylinder on the outside, making the former stationary, as shown on the accompanying drawing. This would simplify the construction, and would do away with the necessity of water-tight pipes for the valve-rod.

Discharge Areas.—To determine the total area of discharge required (which is usually made the same for the filling as for the emptying valves) the following rule may be used:

Let s be the horizontal area of the chamber;

A, the total net area of the valves;

h, the lift between pools;

g, 32.2 feet, the acceleration of gravity;

t, the time of filling or emptying in seconds, all valves being opened fully and simultaneously;

m, the coefficient of contraction, equal to about 0.62.

$$\text{Then } A = \frac{s}{mt}\sqrt{\frac{2h}{g}} = \frac{s}{t \times 0.62} \cdot \frac{\sqrt{h}}{4.01}.$$

In a comparison of a number of locks in America we found that the proportion of net opening for filling or emptying the chambers varied from one square foot in 1800 cubic feet to one square foot in 4000 cubic feet, the first ratio applying to locks of low or moderate lift (up to 9 or 10 feet), and the second to locks of high lift (up to 18 feet). The cubic feet referred to are based on the number of cubic feet of water required for a lockage when both pools are at the crests of their respective dams, and the ratio is therefore independent of the size of the chamber. In certain of the large locks of the lower Seine the proportion is 1 in 4300, and in examples on the Moldau 1 in 3400.

CHAPTER IV.

FIXED DAMS.

General.—While the construction of fixed dams has been practically abandoned abroad for navigation purposes, it still continues in America, and it is in fact only within recent years that movable dams have been applied here, and this application has been limited to two or three streams.

Fixed dams are frequently justified by existing conditions, but they are an obstruction to navigation at periods when there would be sufficient water in the natural state of the stream. They have been built, and are still being built, where the conditions are also favorable for the construction and operation of movable dams, because they can be constructed of cheap materials, usually abundant in the locality, and because they require but little attention for some years after completion, except occasional repairs.

In streams having small commerce the use of fixed dams is not usually objectionable, while for purposes of storing water and furnishing power they are well adapted. This fact led largely to their use in this country, because nearly all the earlier river improvements were made by corporations or by State governments, and one of the chief objects was to secure power for industries as well as water upon which to transport their products. With rare exceptions these streams have come under the control of the United States, and the use of power from their pools has been greatly curtailed or entirely abandoned. The improvements have been frequently extended farther up the rivers for the sole purpose of navigation, and it has been done in all cases by a continuance of the fixed systems already in use; generally the locks have been enlarged and improved, but the dams, whatever their construction, have remained of the fixed type. It has been found necessary to rebuild many of the older ones as they were made of wooden cribs filled with stone, a class of construction which the attacks of floods and drift and the alternate exposure to air and water injure sooner or later, ultimately necessitating extensive repairs. Fixed dams also cause floods to reach a greater height than would otherwise be the case.

Alignment and Length.—Fixed dams are generally placed near the head of the lock, this location being adopted in order to avoid strong currents in the lower approach. They are usually built straight and at right angles to the axis of the river, but other forms may be seen in old structures, especially in Europe, sometimes with a curve of large radius, sometimes with straight or broken arms inclined more or less to the axis of the river, in order to obtain a longer spillway. It is important that a dam of the

fixed type should restrict the waterway as little as possible, and therefore it ought to be as long as the conditions will permit. The greater the length over which the action of the water is extended, the less will be its tendency toward undermining the works and the banks, and the greater will be the facilities for discharging floods.

No rules of any practical value can be formulated for determining the length of a fixed dam, as can be done for movable dams. In theory it should be such that just before the lock is drowned, boats can pass over the dam in either direction; in other words navigation must never be interrupted. Practically, however, this condition is affected by a number of external elements whose combined effects cannot be foretold, such as the rapidity of the rises, the length of the pool below, the width or narrowness of the river in the first mile below the dam, and similar conditions. The only safe rule is to follow that of experience, which shows that the spillway should be as long as practicable.

General Design.—In this country the greater number of stationary dams are built of timber cribs filled with stone, and decked or floored over. This deck is sometimes made a continuous slope from the crest to near the lower pool level, and sometimes is broken into steps of varying heights and widths. The former are known as slope dams, the latter as step dams. Where practicable these cribs should be founded, at least along the face or down-stream side, upon solid rock.

The slope and the step dam have each their advantages and drawbacks. The decking of the latter is more easily injured by the passage of ice, drift, saw-logs, etc., but in moderate stages it retards the progress of the water so that it arrives at the lower pool with less velocity than had it passed over a slope, and hence its effect is not felt so far below it. It is also a little cheaper to construct than a slope dam.

The bottom width of the dam must of course be sufficient to prevent overturning, and its foundation strong enough to resist the pressure that is to come upon it. The ordinary timber and broken-stone dam is usually much wider than its height, one reason for this excess of width being that it gives a more gradual descent to the water from pool to pool, and thus reduces its undermining effect. In one example of old construction, with a lift of 17 feet and a rock foundation, the base width is 43 feet. Another, recently built, with a similar foundation, has a lift of 18 feet and a base of 50 feet, and is 30 feet high. A third has a lift of 14 feet and a base of 33 feet.

The natural foundation will largely govern the design of the dam. If of rock, the case will not present any special difficulty, but if of gravel or other light material, great care must be used to prevent washing and undermining. In cases where the river is deep the cribs usually rest directly on the bottom, but where it is shallow a trench is dredged out, or piles are driven and cut off just below water and the crib-work built upon them. If the last method is used, an apron crib should always be sunk against the down-stream side of the dam as deep as possible, or the reaction may undermine it. Experience has shown that aprons formed simply of piles framed and decked over, or of cribs resting on piles, are unsuited to rivers of high floods, and

View of a Step Dam (Construction just finished).

View of a Slope Dam, with Comb-stick and Ten-foot Apron.

(To face p. 200.)

in more than one case serious damage has resulted from their use. It should always be remembered that the vulnerable part of a fixed dam or of a weir is its down-stream side, and as much care must be taken in protecting it as in securing the up-stream side.

Where the dam is to be upon a pile foundation, in whole or in part, a pile is provided at each of the intersections of the crib timbers, and one row close together along the down-stream face, unless an apron crib is used.

Details of Construction.—The crib dam of modern design is usually built of sawn timbers, 10 inches to 12 inches square, laid crosswise so as to form pens 8 to 12 feet apart, and filled with riprap. The stone is usually of small sizes and irregular shapes, called "one-man stone," but sometimes these are taken out in large blocks as blasted in the quarry, and placed with a derrick. Limestone and sandstone are both utilized, but the latter wears rapidly through the action of water in dams which have leaks through the upper face or spaces in decking. The cost or inaccessibility of stone may sometimes render it necessary to use gravel for filling the cribs, but this usually contains so many small particles that will be washed away that its use is not desirable.

The timber generally used is white oak or yellow pine, but any of the heavier woods may be employed, if under the low-water line, or where they will always be moist. It may be either sawn, hewn, or left in its natural state so far as the cribs themselves are concerned, but the deck or floor covering the whole structure should be either sawn or hewn. At the present time hewn and round timbers are very rarely used, as sawn timbers can usually be obtained for a little extra price, and are much preferable for rapidity of construction. The pieces may be laid directly upon each other, or daps may be cut at their intersections, at which points they are drift-bolted. The former method, however, is as satisfactory in practice as the latter, and less expensive.

The timbers which lie in a direction across the stream are called stringers; those which have a direction parallel to the current are called ties. Both sets of timbers should have a length as great as practicable, economy being considered, and the joints may either butt or be spliced. If the former, a block of similar section to the stringer or ties should support the joint and lap onto each timber from 1′ 6″ to 2 feet, drift-bolts being used to connect the block with the stick below as well as with the one above. These joints should not appear immediately over each other in two succeeding layers of timber. If the joint is spliced, it may come at the intersection of the timbers with those lying in the opposite direction; if it does not, then it should also be supported by a block, although this is rarely done. The drift-bolts used are usually $\frac{5}{8}''$ or $\frac{3}{4}''$ square, depending on the size of the timbers, with heads and with wedge points. Round or nail points should not be used, as they split the timbers, and square drift-bolts are preferable to round ones as they hold better.

Both the up-stream and down-stream sides, known as the back and face of the dam respectively, are carried up vertically, the former being covered with a single or double row of sheet-piling, driven as deep as possible, and which should extend to the

top or near the top of the cribwork, and there connect with the decking of the upper slope. Some engineers cut it off about 2 feet below the top, and continue it with short plank, for easier repairing. Immediately up stream of this sheeting should be placed an embankment of gravel and clay, riprapped on its upper surface for a distance of 10 or 15 feet away from the dam. The down-stream face may be left open between the timbers unless the spaces exceed 7 or 8 inches, when they should be reduced by fillers, or the filling stone will be washed out. The up-stream face may be carried to within 3 or 4 feet of crest height; the lower face should stop at or near lower pool level.

The line of the crest is ordinarily placed from 8 to 12 feet from the upper face, and the sloping decking connecting these points should be practically water-tight. It is placed on a slope to permit the passage of drift, etc.

If the dam is of the step type, the height of each step should not much exceed one-third the width of the next below, or in certain stages the falling water may miss the step and strike the one below, and if drift or ice is running, the decking will suffer accordingly. Thus, where the steps are 10 feet wide, the limiting height should be about 3 feet 4 inches. The step descending to the apron, however, if the latter is wide, may be as high as 5 feet. The width of these successive steps may vary between 8 and 12 feet, 10 feet being usual. The decking of the up-stream slope should be of two thicknesses of 2-inch or 3-inch white oak or hardwood plank, lap-jointed so that when the top layer is worn it can be renewed without destroying the lower one. The decking of the other steps may be from 4 to 10 inches in thickness, depending largely upon the amount of ice, drift, etc., passing over the dam, which soon wear away the surfaces.

In certain cases the top of the cribwork at the crest is stopped from 6 to 12 inches below the upper pool, and the remaining height obtained by spiking a "comb-stick" on the decking. This stick should be of oak, and be well secured with drift-bolts and straps, as it has to stand heavy blows from drift. It is frequently necessary to use this method in order to level up or raise the crest of a dam which has settled, which always happens as the timbers become old, and in point of fact the settlement usually dates from the completion of the work.

In rivers with small flood heights, especially where it is desired to keep the pool at about the same level, flash-boards are often used; these are plank laid horizontally along the crest and supported against pins. As the river rises one or more planks are removed, and replaced as it falls, thus securing a regulation of the pool.

Where slope dams are built the up-stream side is made as for a step dam, and the down-stream side is made with a slope varying from 2′ 6″ to 4 feet base to 1 foot rise. As generally built these dams provide for but little protection below, but it has been found necessary in almost every case to place apron cribs below them later in order to prevent undermining. Where this method is adopted in the first case, as it always should be, the cross-section of the dam itself may be reduced, as the apron can be made part of the structure.

General View of Dam No. 7, Kentucky River, Ky., U. S. A., During Construction.
The dam is of the step type and built of timber cribs filled with riprap. The length is 350 feet, with a base width of 60 feet. The lift is 15½ feet. The dam was completed in 1897

(To face p. 203.)

The apron is occasionally made with a considerable upward inclination in the down-stream direction, this construction having been found advantageous in causing deposit along the down-stream face of the dam.

The width of the apron crib, which in a step dam is formed by the lowest step, is dependent on the depth of water below it and on the lift, since the higher the lift the greater will be the width required. It may be determined on the proportion of 18 inches of width for each foot of lift, with a minimum of 8 or 10 feet. Thus for a dam of 4 feet lift the apron would be 8 or 10 feet wide, while for a dam of 12 feet lift it should be 18 feet or more. After a flood has reached a depth of 4 or 5 feet on the crest, the profile of overflow becomes the same for slope as for step dams, and a violent reaction occurs where it strikes the lower pool, and not infrequently pieces of the apron decking are loosened by it and sometimes a large number torn up at once. Especial care should therefore be taken to fasten the top stringers to the stringers below, and it is best to make the former of oak or hardwood, which will hold fast the drift-bolts of the decking. If these precautions are taken, pieces will rarely come loose. The decking of the steps or slope above seems to remain practically unaffected by the movement of the water.

The deck timbers on the down-stream side of a slope dam and on the steps of a step dam are sometimes spaced one-half inch to one inch apart, in order to permit a free circulation of air and the quicker escape of the overflowing water. This, however, is of little practical value, since it has no appreciable effect on the discharge, and it has been found that it is liable to wear away rapidly the filling underneath. With the apron decking, however, it is considered best to space the timbers a little apart, as it is supposed to check the reaction in ordinary stages, and to reduce the tendency to loosen the timbers. Where the reaction has been violent, however, open-spaced decking has been torn off, notwithstanding the precaution of placing the pieces an inch or more apart.

Methods of Putting in—Without Coffer.—Crib dams are usually constructed without the use of a coffer-dam, during the low-water season. This feature constitutes one of their chief recommendations, as it permits of cheap and rapid construction. The site is first cleared, either by dredging or otherwise, of all things which might interfere with the proper seating of the cribs, and then carefully sounded throughout in order to determine the location of the various irregularities. Its length is then divided into crib sections of such dimensions as it is considered can be handled in the current with the appliances available. These may be up to 100 feet in length and of the whole width of dam, and in practice they are generally put together immediately over the place where it is proposed to sink them; but this may be done, or partly done, elsewhere, and they may be towed into position when ready. Having determined by the soundings the profile of the bed, the under side of the crib is shaped approximately to fit it by means of additional timbers or blocks, drift-bolted on where required, so that when the crib reaches the bottom its top will be nearly level. The cribs are

built in the water, sinking as timber is added until the bed is reached. They should then be weighted with stone on top to settle them. The construction may begin at one or both sides of the river. Only sufficient stone is immediately filled in to hold the crib in position, as it is desirable to allow the water to pass through with as great freedom as possible until the dam is nearly completed. As soon as one crib is built up to a height somewhat above the water stage, and is safely anchored, another is sunk at its outer end until all are in place. The discharge area of the river will have been contracted considerably by the time the last crib is put in, and the velocity increased, so that it may be necessary to carry anchorages or wire ropes up stream to prevent too great displacement. Should slight displacement occur, however, it is usually not of serious import, as the timbers above water-level can all be carried through on the proper line. If the dam is on rock, with therefore no danger of the river cutting under the cribs, the work of connecting them by a continuous crib covering the several joints should be vigorously carried forward. During this time there should be on hand an abundant quantity of stone which can upon short notice be dumped into the cribs in case of an approaching rise, some of which may be put in place as the work progresses. When the cribs have reached their full height the filling should be rapidly completed, and after that the decking may be placed. Finally, after all has been done and satisfactory connection made with the lock wall and abutment at the ends, the sheet-piling along the upper face must be put in place. This should invariably be done with great care and should extend to the rock whenever practicable. As this approaches completion the water above will rise and flow through the valves in the lock, and if the discharge of the river exceeds the capacity of these valves it will eventually rise over the crest of the dam. It is, therefore, necessary to proceed rapidly with the sheet-piling, and this portion of the work should be carried on continuously and at several points. Where there is a considerable flow of water it may be necessary to open the lock gates and depend upon the erection of the coffer-dam in the head of lock to close them again with safety. This is not advisable, however, unless the conditions for dry weather are favorable, as the appearance of a sudden rise might render it impracticable to place the coffer in position, and the increased flow passing through the lock might endanger its safety. When the sheet-piling has been completed an embankment of gravel or other suitable material should be placed immediately above, as has been mentioned, and protected by broken stone to prevent its being washed over the dam.

Where the foundation is of light material, so that there would be danger of undermining before the sheet-piling could be put in as just described, the latter should be driven as soon as the foundation is above water, and gravel and brush should be kept on hand to stop any under-cutting. In this case the piling is sawed off at the water surface and the river flows over it, and when the dam is finished the spaces above are closed by a double row of planks resting on the sheet-piles and spiked to the crib timbers. This method is usually to be preferred in all cases where the dam does not rest on bed-rock.

With Coffer-dam.—There are cases, as in the construction of a masonry dam, in which it is necessary to perform the work inside a coffer-dam, or rather inside two coffer-dams, since it is not practicable to close the whole opening at one time in a stream of good discharge. The type of coffer to be used will depend upon the character of foundation upon which it is to be built and the style of dam proposed. The different kinds in use have been described in the chapter on "General Designs."

If desired, the coffer may be so placed as to become a part of the completed structure, or it may be left to form a protection, the down-stream arm being reduced to the proper height and floored over for this purpose. The walls of the coffer are carried out from either the abutment or lock, generally from the one having the poorest foundation, so as to throw the increased current caused by the contraction over the best foundation. After they have reached the middle of the river, or such a point as desired, they are connected by a cross wall, extending in the direction of the stream. The inclosure thus formed, when pumped out, will constitute the building ground for the dam. When the dam has been completed as far as desired within this inclosure, a bulk-head reaching several feet above crest level is built near its outer end, connecting the two coffer-walls. This done, the latter may be removed, and the second section of coffer for the opposite part of dam, which should be considerably higher than the crest of dam, on the up-stream side, may be put in and pumped out, and the work proceeded with.

The discharge of the river during the latter part of the construction may be passed through the lock, or through sluices left in the dam, or it may flow over the unfinished surfaces of the dam, each side of the latter being built up alternately a foot or two at a time, using needles or stop-planks to keep the water off the masonry being placed.

Abutment and Protection of Bank.—The abutment and the protection above and below it should be completed before much work is done on the dam, or the river may cut into the exposed bank. The abutment must be provided with a wing wall of construction similar to those for the lock, connected with the solid bank by sheet-piling or by a crib sheathed inside and filled with tamped clay. The wing should be well puddled, and where the soil is light the sheet-piling should be continued for some distance back from the masonry and driven as deep as possible. The top of the abutment is usually made level with the top of the lock walls.

The construction of an abutment is frequently a difficult problem, since it may have to be placed deep in the bank, and the bank may be full of seepages. One method of overcoming this is to drive sheet or round piling to form a rough coffer-dam against the earth, and then to excavate as rapidly as possible inside the inclosure, and build up a foundation of concrete to the height that may be required. If the work is done quickly the concrete will be in place before much more slipping has occurred, and will permit the remainder of the masonry to be put in with more leisure. The piling should be driven a little distance from the building lines, as the earth will push it in

more or less during the excavation. If the masonry is to rest on piles, they should be driven before the excavation is begun so as to save any delays.

Another method is to excavate as deeply as possible without using sheet-piling, and then to take sharpened planks, and set them up in a close row against "rangers," or horizontal planks, which are kept in position by shores or horizontal struts, each side of the excavation being braced against the opposite side. The material is then dug out below the feet of the vertical planks, which are driven down by hand as far as possible, when the material can be removed from the center of the excavation. Leaks may be checked by using straw, weeds, excelsior, or similar materials. This operation is repeated till the final depth is reached, when the masonry can be put in as above described.

In a bank of very unstable nature it is best to make the finished grade to a flat slope, two or three to one, and unless the seepage is very bad, it will be found that the bank will drain itself and become stable. It must of course be paved or riprapped, and its foot for 150 or 200 feet below the abutment should be provided with a toe made of a crib filled with riprap, or, better still, built of concrete. This should always go to rock, or, if the foundation is of gravel, it should be sunk as deep as possible and have a close row of long piles driven along its outer side, faced with heavy riprap. The bank above this crib and around the abutment should be paved with blocks with close joints, or with hand-placed riprap, and if the river is subject to high floods it is well to bed the stones roughly in Portland-cement mortar, where they will be most exposed to the reaction from the dam.

Where the range of floods is very small the precautions need not be so elaborate, and in such cases a protection of piles and fascines, or of woven brush and riprap, may be sufficient below the abutment.

Too much care can hardly be taken in securing this end of the dam, as several cases have occurred where the river in a single flood has cut away the banks and found its way around the abutment.

In most cases it is necessary eventually to continue the protection of the banks, both below the lock and below the abutment, until it has reached a length of many hundred feet. As a rule, it will be sufficient for the first season to rely on the riprap or paving below the abutment as just described; but if the banks are "rotten," that is, if they are sandy and liable to slips, a good supply of stone should be kept on hand for emergencies. In the following season the bank, where cut away by the water, should be graded and riprapped, and this process should be ultimately continued as far down stream as the wash and eddies from the dam show to be needful. This will range from 200 to 1200 feet or more, depending on the lift of the dam and other circumstances. A rough practical rule may be made that for each foot of lift of a fixed dam 100 feet of bank will be affected by the washing, and for each foot of lift of a movable dam, 150 feet.

Unless the bank is very hard, the riprap should never be placed on the bare soil, but always on a bed of spalls or large gravel, 6 inches or more in depth. Where this

General View of Dam No. 1, Barren River, Ky.

The dam is of the slope type with a comb-stick, and is built of timber cribs filled with riprap. The length is 268 feet, with a base width of 80 feet, and a lift of 15·1 feet. The original structure was completed about 1840, but has been practically rebuilt several times since then.

(To face p. 206.)

latter covering is omitted the ripples and waves from the dam, which form the agency by which the banks are cut, play through the openings in the stones and wash upon the soil with a force very little diminished, and soon carry away the particles of earth. We have seen the two methods tried side by side upon a bank just below a dam, and the portion protected by riprap alone was eaten away almost as though no stone lay upon it, while the portion on which spalls had been placed was unaffected.

The protection of the banks is a matter which is usually neglected until some action is absolutely necessary. This is unfortunate, as by that time the bank has usually become badly washed and must always remain an eyesore. Had the remedy been applied at the proper time, the ultimate expense would have been no greater and the disfigurement of the property would have been avoided.

Several notable examples of the results of lack of bank protection are to be seen in this country. On one river the water at some of the locks has eaten into the bank till it has excavated an area of several acres, below and behind the land wall, which in one case resulted in the river washing its way during a flood around the upper wing wall into the basin and causing very serious damage.

Masonry Dams.—Several examples of masonry dams of concrete or of stone are to be found in rivers in this country, while still others are proposed. Where the foundation is good they are preferable to timber dams because of their durability, although they are of course more expensive. The principles governing their design and construction are the same as those for storage reservoirs, except that the effects of the overflow must be provided for. They should always be composed of concrete or of large stone laid in good cement mortar, so as to secure a structure which will be water-tight and stand the attacks of drift and floods. A dam of this class built with unsuitable materials is a source of constant trouble and expense, and may eventually have to be removed and rebuilt.

COST OF FIXED DAMS.

Location.	Lift.	Length	Width of Base.	Average Height.	Approx. Contents Cu. Yds.	Cost per Cu. Yd.	Total Cost	Cost per Foot Run	Remarks.
Kanawha River, W Va., Dam No. 2, 1887	12′ 0″	524′	38′	20′	14,750	$6.35	$94,000 00	$180.00	Cost of dam only, of cribwork and stone filling.
Kanawha River, W Va., Dam No. 2, 1887	.....	....	..	....	....	7.58	111,500 00	212.00	Cost, including abutment, bank protection, etc.
Fox River, Wis., Princeton Dam, 1897	about 4′	180′	30′	10′	1,420	7 80	11,083 00	61.50	Cost, including abutment, bank protection, etc. Dam is of cribwork and stone filling. Cost of dam only, $3 70 per cu. yd.
Fox River, Wis., Berlin Dam, 1893	3′ 4″	200′	30′	10′	1,580	9 35	14,747.00	73 70	Cost, including abutment, bank protection, etc. Same type as at Princeton.
Kentucky River, Ky., Dam No. 7, 1897	15′ 0″	351′	60′	20′	13,000	2.00	25,780.00	73 65	Cost of dam only, of cribwork and stone filling
Green River, Ky., Dam No. 5, 1899	14′ 0″	281′	46′	16′	7,000	3 35	22,780 00	81 90	Cost of dam only, of cribwork and stone filling.

CHAPTER V.

MOVABLE DAMS.

History.—A movable dam differs from a fixed dam in that the dam proper is so designed that it can be lowered or raised as may be needed, its principal object being to provide slack-water in times of low water, without forming an obstruction to navigation in moderate stages, or to floods. The last is an advantage of great importance on rivers of low banks, as a movable dam does not increase the dangers of inundation.

With the exception of the bear-trap dam, which is an American type, the invention of movable dams is due to the genius of the French engineers. Prior to 1830 the fixed or stationary dam was the only one used for navigation purposes. These had been in use on the Lot since the thirteenth century, and, with the introduction of locks in the fifteenth century, had been constructed on many rivers, but they were open to the same objection that exists to-day—the principle is in part unfavorable to navigation. Dams with small navigable passes had been used, but the ascent of the pass was always very laborious. These openings or passes were closed either by beams lying one upon another, supported against piles or piers at the ends, or by planks resting against a sill in the river-bed at the bottom, and against a beam spanning the opening at the top. When it was desired to open the passage the beams or planks were removed, either one by one, or simultaneously, and the water rushed through with great violence. Sometimes they were used for the purpose of producing artificial floods, by damming up the whole river until the level of the pool above the dam had been raised to the desired height, when, by their sudden removal, the water escaped and carried rafts or boats over the shallow places below. The operation of letting out the water was called "flushing" or "flashing"; in this country on log streams it is called "splashing."

On the Yonne and one or two other rivers in France this system of navigation was still in use recently, the flashes being commenced at prearranged times at the upper end of the river and continued down stream successively as the boats neared the dams. The passes were 25 to 40 feet wide.

Falling gates or shutters, supported by props when upright, were built across a fixed dam on the river Orb, in France, in the eighteenth century, forming the first attempt at placing movable weirs on fixed dams.

NOTE.—Part of the matter in the following chapters, with some of the illustrations, is taken from a paper on "Movable Dams," by B. F. Thomas, published in the Transactions of the American Society of Civil Engineers, June, 1898, and republished by permission of that Society, and others of the illustrations are republished by permission from "The Design and Construction of Dams," by Edward Wegmann, C.E.

The first distinct type of movable dam was erected in 1818 on the Lehigh River, in the United States, and was called a bear-trap dam. It consisted of two wooden gates revolving on horizontal axes at the floor level. The down-stream gate pointed up stream, and the up-stream one pointed down stream, the latter resting on the edge of the down-stream gate when raised. The dam was operated by water running under the gates through culverts and forcing them up. The example was not copied, and until recent years the type remained practically unknown.

In the year 1834 M. Poirée, an eminent French engineer, invented the needle dam. This ushered in a new era in navigation, and this type of dam soon multiplied and was improved and modified, and other inventors came forward with new ideas, some good, some bad, until to-day there are numerous systems from which to choose.

The invention of movable dams was only arrived at after long discussion of ways and means for more successfully operating the apparatus used for closing the chutes in the old stationary dams. The use of needles was then already old. The problem to be solved was how best to widen the passages to accommodate the increased requirements. The experiment of supporting the tops of the needles by a rope was tried, and was in a measure satisfactory for lifts of 2 to 3 feet on passes of considerable width. The rope, which was braced to the down-stream side of the foundation by strips of wood, was tied to a pier by one end while the other was wound on a windlass. To open the dam it was only necessary to release the line at the pier, when the whole set of needles would float out, being attached to the rope beforehand, as were also the braces.

This was the status of improvements in fixed dams when the first movable dam was constructed, and it was only natural that iron trestles should supersede ropes in needle dams, that gates sliding on these trestles should later on replace the sluice-gates of the old chutes operated from an overhead bridge, that the swinging wickets formerly used to increase the heights of stationary dams should form the dam itself in after years, and that *poutrelles* hinged together and supported on trestles should form the curtain dam that was to come into use.

Classes and Kinds.—Movable dams may be divided into two general classes: (1) those requiring extraneous power for their maneuvers, and (2) those operated by the force of the water. Among the first class may be named the various types of trestle and wicket dam, like the Poirée, Chanoine, Boulé, Cameré, etc., while the second class comprises the several forms of bear-trap, drum-wickets, etc. The first class is practically the only one so far applied to navigable rivers, and its application until recent years has been confined largely to the wickets of Chanoine and the trestles and needles of Poirée.

The forms of closing are many, and frequently vary on the same dam; for instance, the pass may be of wickets and the weir of needles, or the reverse may be the case; while more than one type has been applied, even on the same part of a dam; for instance at the Suresnes dam, in France, the pass is closed by trestles supporting alternate bays of Boulé gates and Cameré curtains.

In the needle dam the water is dammed up by vertical planks, called needles, resting against bars connecting the trestles at the top, and against a sill in the river-bed at the bottom. The trestles are spaced from 3 to 8 feet apart, and when not in use lie down across the stream, being protected from injury by the sill. A walkway connects them when standing.

The Chanoine wicket is an upright shutter, hinged near its middle to a horse connected to the floor, and also to a prop which rests against a shoe on the floor. The displacement of the end of the prop permits the wicket to fall with the current.

The Boulé gate and Cameré curtain replace the needles by small gates and by curtains respectively, resting directly against the trestles or against uprights leaning on the trestles.

In the overhead bridge dam, where the closure is always composed of such gates or curtains, the supports are all drawn up to the bridge when not in use, while their bottoms rest against a sill in the river-bed when in use.

These are the principal types of dams, and practically the only ones employed. Many modifications of each one have been suggested, but rarely, if ever, put into practice, and will therefore not be described.*

Height of Lift.—The lift of movable dams has, until recent years, been very moderate. It was the idea of the earlier engineers, who did not possess the appliances of the present day, that all parts must be small enough to be maneuvered by hand, and in consequence the head of water had to be small. Thus on the first needle dams the head was limited to about 4 feet. As experience was gained, however, the lifts were gradually increased, until we find examples to-day of needle dams with lifts of more than 10 feet, and a curtain dam at Poses, on the lower Seine, with a lift of nearly 14 feet. Several Boulé dams are also in existence with lifts of 10 to 12 feet.

The lift of a Chanoine dam, however, has rarely exceeded 8 feet. No recent examples of this type are to be found in Europe, where other methods of closure have come into use, and in consequence the Chanoine wicket has never received its proper development. There is no reason, however, why it cannot be used for much greater lifts than heretofore, since it simply means the application of heavier parts to hold back the water, and heavier machinery with which to perform the operations, than are now used.

The greatest drawback of movable dams has been that their cost was far too great for the amount of river made navigable thereby; in other words, the lift attained was too small to justify the expense, and their success cannot be considered complete until they have been applied to lifts at least equal to those which would have been given to fixed dams at the same points, and at not much greater cost.

General Design.— The designing of a movable dam, in order to secure the best results, is one of the difficult problems of engineering, and requires not only a thorough

* Most of these modifications, such as the Stickney Lifting Dam, the Marshall Bear-traps, the Janicki Folding Dam, the Venable Tunnel Dam, etc., will be found described in the paper on "Movable Dams," Transactions of the American Society of Civil Engineers, June, 1898.

knowledge of construction but also a practical acquaintance with the operation and effects of movable dams in all their bearings.

Every such dam not operated by the natural forces of the water should fulfill, as far as possible, the following conditions:

(1) The head of water sustained should not be less than that advisable for a stationary dam at the same point.

(2) The dam should be capable of being operated by the regular employees and appliances, both in lowering and raising, under full head, in whole or in part, without much risk to the operatives.

(3) The crest should be submersible to a sufficient extent to regulate the flow at ordinary stages.

(4) The leakage should not exceed the discharge of the stream at any season.

(5) The parts should be complete in themselves without the introduction of additional means for sustaining the water, even in low-water seasons.

(6) The cost should not greatly exceed that of a fixed dam for the same location.

In general, movable dams are constructed in two or more sections, one for navigation, called the pass, and one or more for the passage of surplus water, called the weir. The object of the latter is to provide a means for passing small rises without having to handle the heavy appliances of the pass. On wide rivers with two or more weirs to the dam they are usually made of different heights, affording more or less easy maneuvering. When the flood has reached the full discharge capacity of the weir, and is still rising, the pass must be lowered, and if the dam has been properly designed there will then be enough depth of water on the pass sill to allow full-draught boats to cross it in either direction; that is, the contracted current must not be swift enough to embarrass up-going craft.

The pass is always located next the lock, so as to throw the currents created by the weir as far from the entrances as possible. It is usually placed opposite the lower end of the chamber, but where the walls are long it is sometimes placed near the middle.

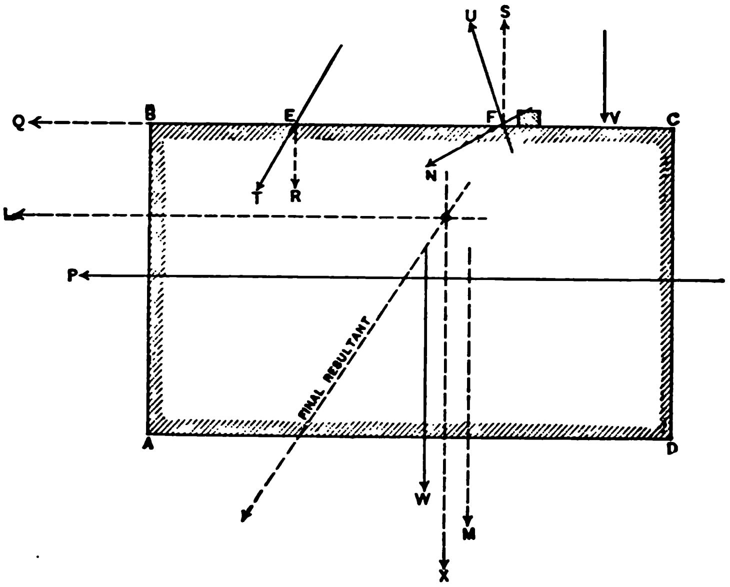


Fig. 13a.

The choice of type is usually decided by the characteristics of the river, and the results of experience in America and in Europe seem to show that for rivers of slow rises needles, gates, or curtains are well adapted, the first

named being also used on small rivers of quick rises and where the low-water discharge is small. In this case the weir is usually closed with Chanoine wickets, although needles are also used. For large rivers subject to high floods the Chanoine wicket has been almost always adopted, and on the whole seems best adapted to such streams. Bear-traps and drum-shutters have not been used except as weirs to regulate the pools.

The calculations for the foundation of a movable dam are based on the same general principles as given for lock-walls, combined with the strains from the movable parts. Thus let $ABCD$ (Fig. 13*a*) represent a section of a foundation for a needle dam. The forces acting per foot run are:

P = pressure of water on CD, CD in this instance being supposed vertical.
V = vertical water-pressure on the apron above the sill.
N = pressure from the needles against the sill.
T = thrust from down-stream leg of trestle.
U = upward pull from up-stream leg of trestle.
W = weight of masonry.

As mentioned in the chapter on locks, W must be calculated with loss of weight by immersion or otherwise, according to the material of the natural foundation. The pressure V will be a function of the difference between pools, if the lower pool is above BF. In this case the lower pool will also cause a downward pressure on BF, which may be combined with V.

Resolve T, N, and U into vertical and horizontal components, assuming N to act upon the same axis as T and U. We thus obtain a horizontal force Q, acting along FB, a downward force R, acting at E, and an upward force S (the algebraic sum of the vertical components of N and U), acting at F.

Combining Q and P we obtain as their resultant the force L, and combining R, V, and W, we obtain their resultant M. Lastly, combining S and M we obtain the force X.

We thus have finally to deal with the two forces L and X, whose resultant must lie within a certain zone of the base as indicated in the calculations for lock-walls.

Similar methods can be applied for calculating the foundations for other types of movable dams.

It should be noted that the water-pressure P acts a little below the middle of CD, as a graphic representation of its elements would be a quadrilateral.

Elevation of Sills.—The depth of the pass sill below water is decided by the mean position of the river-bed and the navigable depth desired. Its top may be placed on the line which will connect the crests of the nearest bars above and below, but must not be higher, otherwise it will itself become an obstruction at the stage occurring just previously to that at which the dam is to be raised, when the lock is out of service, a condition quite common during the winter and spring, when it is not always advisable to raise the dam on account of ice or the approach of floods.

The sill of the weir is usually placed higher than that of the pass and its elevation depends on its relation to the length of spillway, as shown in the next paragraph. Where several weirs are employed on the same dam, their sills usually conform approximately to the natural bed of the river.

Length of Pass and Weir. — In determining the lengths required, the chief conditions to be satisfied are that the pass shall be long enough to permit ample room for tows going through, and that this length, combined with the elevation of the sill, shall be such that there will be only a very slight fall or "swell-head" there when the navigable depth on it is at a minimum. The last condition is an important one, as, if there is a fall of more than a few inches the current will prove too strong for ascending boats.

To illustrate the methods used in solving the problem an example from actual practice is given. In this case the pass was assumed as 130 feet long and the "swell-head" was assumed as 0.5 foot. The lock, which was assumed to be opened during floods, thus assisting in the discharge, was 52 feet wide. After construction the swell-head was found to be about 0.4 foot, the lock gates being kept closed except when deposit had to be washed out of the chamber.

The following calculations were made to determine the principal dimensions:*

"Assuming a value of 0.5 foot for the swell-head, the question then is to determine the maximum stage of the river, the discharge corresponding to which will pass through the pass and lock without causing an increase in depth of water above the dam over that below the dam exceeding 0.5 foot. This will give the elevation of the sill of the weir above the sill of the pass.

"To determine this the Chanoine formula is used, to wit:

$$Q = M(LH + L'H')\sqrt{2g(Z+h)},$$

in which Q = discharge of river in cubic feet per second;
M = constant depending on stage;
L = length of pass in feet;
L' = length of weir in feet;
H = height of water below dam above sill of pass = stage in this case;
H' = height of water below dam above sill of weir;
h = height due to observed velocity of approach $= \dfrac{v^2}{2g}$;
g = 32.2 feet;
Z = swell-head, as explained above.

"For any stage H the discharge through the pass will be $Q' = M(130H)\sqrt{2g(Z+h)}$, and through the lock $Q'' = M[52(H-1.25)]\sqrt{2g(Z+h)}$, as the upper miter-sill is 1.25 feet higher than the sill of the pass; so that for the pass and lock $Q = Q' + Q'' = M(182H - 65)\sqrt{2g(Z+h)}$. For $H = 5$ feet, $M = .703$, $v = 2.72$ feet, and the equa-

* Annual Report Chief of Engineers, U. S. A., 1897, p. 2538.

tion becomes $Q = .703 \times 845 \times 8.02 \times .784 = 3735$ cubic feet, which is considerably greater than the discharge corresponding to a 5-foot stage. Making H in the formula equal to 6 feet, M becomes equal to .71, $V = 2.99$ feet, and there results $Q = .71 \times 1027 \times 8.02 \times .8 = 4678$, for the discharge by the lock and pass with a swell-head of 0.5 foot. If Z is taken as equal to 0.5 foot, and the sill of the weir is made 6 feet above that of the pass, H becomes 0.5 foot, and M is reduced accordingly. Taking L' equal to 140 feet, the discharge over the weir becomes equal to 180 cubic feet per second, so that the quantity of water that can pass the dam per second, the lock gates being open, without producing a greater swell-head than 0.5 foot, is 4858 cubic feet. The discharge of the river corresponding to a 6-foot stage is 4910 cubic feet per second, so that the swell-head corresponding to this discharge would be but little in excess of 0.5 foot.

"In determining the length of the weir the following conditions must be satisfied: 1st. The area of discharge afforded by the weir must be sufficient, when taken in connection with those of the pass and lock, to permit the passage of discharges corresponding to all stages up to the level of the top of pier and abutment, without causing a greater swell-head than 0.5 foot. 2d. The discharge area of the weir should be sufficient to pass all discharges corresponding to stages up to that at which the natural river is navigable, without the removal of any needles from the pass. It may be stated, however, that the second condition will be satisfied by a length of weir that will satisfy the first.

"To determine the length of the weir, the elevation of its sill being fixed, the formula of Chanoine, modified to take into account the discharge through the lock, is used, to wit:

"$Q = M[LH + 52(H - 1.25) + L'H'] \sqrt{2g(Z + h)}$, or substituting for L its value already determined, 130 feet, the formula becomes $Q = M(182H - 65 + L'H') \sqrt{2g(Z + h)}$; making H equal to 7 feet, H' becomes equal to 1 foot, and the formula becomes $Q = M(1209 + L') \sqrt{2g(Z + h)}$. For a stage of 7 feet, $Q = 6362$ cubic feet, $M = .73$, $h = \frac{v^2}{2g} = \frac{(3.21)^2}{2g} = .160$ and $(Z + h)^{1/2} = .812$, whence L' becomes equal to 129.3 feet.

"For high stages, the formula of Chanoine and De Lagréné is used, to wit: $Z = 1.5V^2\left(\frac{S^2}{S'^2} - 1\right)\frac{1}{2g}$, in which Z = swell-head, V = mean velocity before construction of works, S = discharge area of river before construction of works, S' = discharge area after construction of works $= LH + L'H'$, and 1.5 is a constant for cases where the lock gates are open. Transforming the above equation and substituting for $2g$ and Z their values 64.3 and .5, respectively, there results $S' = \left(\frac{1.5V^2S^2}{32.2 + 1.5V^2}\right)^{1/2}$. For the highest stage observed $H = 15.45$ feet, $H' = 9.45$ feet, $V = 4.7$ feet, and $S = 4576$ square feet, and $S' = LH + L'H' = 3260$ square feet, $= 2008.5$ square feet $+ 9.45L'$, whence $L' = 132.4$ feet.

"As the stage that would just cover the pier and abutment is 16.5 feet, it is thought best to fix the length of the weir at 140 feet, particularly as the rises during the summer are generally quite sudden.

"With the sill of the pass at low-water level, and 130 feet long; the sill of the weir 6 feet above that of the pass, and 140 feet long; for the 8-foot stage Z becomes equal to 0.53 foot, for the 9-foot stage it is less than 0.5 foot, and when the pier and abutment are submerged Z will not exceed 0.5 foot appreciably."

Foundations, etc.—The foundation for a movable dam should be of masonry, and wherever practicable should be begun on bed-rock; where this is not feasible substantial aprons or other protection must be provided for the down-stream face. This type of dam, owing to the regulation of the pool, is more subject to the dangers of currents and reactions than a fixed dam, and should any settlement or undermining occur it may throw the whole superstructure out of order. For this reason the substructure should be made secure and of permanent material, as, if repairs have to be made to it later, it will necessitate expensive coffer-dams and probably a long interruption of navigation. A fixed dam will safely stand a settlement which would be dangerous to a movable dam, and can, moreover, be much more easily repaired. The long experience of European engineers in this field has shown the wisdom of using nothing but permanent foundations for movable dams, and they discarded long since the use of timber for such purposes.

Where the coping of the masonry is of cut stone the two upper courses should be doweled together, and in many cases long bolts are run horizontally from the up-stream to the down-stream face. Especial care must be taken to make the former proof against undermining, since there is no mass of backing to prevent leakage, as with a fixed dam.

The sills supporting the bottom of the curtain of the dam should be made preferably of cast iron, although where always under water wood has been generally employed in this country. Cast iron is, however, much preferable, since the wood becomes gradually worn away by the currents and can only be replaced with difficulty and expense. The depth behind the sill, or the recess, should be sufficient to allow all movable parts to lie down, with an inch or two of clearance above them, so that any submerged object will strike the sill rather than the parts below. The depth of these recesses varies from 15 inches to 20 inches or more, one recent example of a Boulé dam having a depth of 3½ feet. It should, however, always be made as small as possible, since it acts as a catch-basin for gravel, sunken drift, etc.

The height of the sill above the masonry on the upper side is from 4 to 6 inches, as may be required to support the ends of the needles or wickets. It is made small, to prevent gravel and loose stone catching against the sill when the dam is down, which would interfere with the raising.

Recesses should be provided in the coping of the masonry into which uprights can be placed for use in coffering off any portion of the dam for repairs to the superstructure.

Drift-chute and Regulating-weir.—It is very desirable, particularly in high dams,

to provide a regulating-weir whose closing apparatus shall always be under control, and which can be maneuvered with certainty, night or day, by a watchman. The proper location for such a weir is away from the lock, so that the water passing through it will not disturb navigation. It should, therefore, be near the abutment, and may be separated from the main dam by a masonry pier. This regulating-weir should be of sufficient length and depth to safely pass all the drift usually found upon the stream up to the stage of water at which the dam is lowered, and to discharge the surplus water at medium stages, but it should not be of such dimensions as to render its maneuvers uncertain.

Abutment and Protection of Banks.—The banks below the abutment and the lock must be protected as described in the chapter on Fixed Dams, since they are similarly exposed to washing, and the design and construction of the abutment should be based on similar principles.

Remarks.—Movable dams, although solving in principle the problem of slack-watering rivers, are open to several objections in comparison with fixed dams, the chief of which are greater expense in establishment, and sometimes in operation and in maintenance, and greater liability to injury from drift and ice, especially on American rivers. The problem of drift has been overcome so far without serious injury resulting, by constant watchfulness on the part of the dam-tenders, and as the districts along the rivers become settled and the timber cleared away, this danger will gradually decrease. The problem of ice, however, will always remaiu a serious one, as the dams have to be lowered when it begins to run, or the drifting pieces will catch in the trestles and other parts, and pile up until lowering becomes impossible, and the force of the river will eventually wreck the dam. Because of this, movable dams on northern rivers have always to be lowered at the approach of winter and kept down until the spring, when the danger is past. Should the season be one of low water, navigation has to stop, the dam being then of no benefit. In certain cases, as where the dam is just below a city, additional inconveniences will result, because all harbor navigation comes to a standstill, and factories which are largely dependent on the river for coal or other supplies, are put to much expense in procuring them elsewhere. This state of affairs has happened more than once with the Davis Island dam on the Ohio, just below Pittsburg, and on certain occasions, at the urgent insistence of the manufacturing interests, the wickets have been raised in winter-time in order to provide sufficient depth for harbor navigation. The result, however, has usually been disastrous, as ice and high water coming suddenly have wrought havoc with the dam.

When some means have been devised of overcoming such a disadvantage movable dams will be of greater benefit than they are now. The problem will in all likelihood be solved by the adoption of a closure of the bear-trap or drum type, which can be operated from the shore and by the natural forces present, and which will allow a free overflow along the crest for drift and ice, with no parts below in which these can become entangled.

View of Weir of Dam No. 6, Kanawha River, W. Va., Illustrating the Effect of Drift on Trestles of Movable Dams.

(To face p. 216.)

CHAPTER VI.

NEEDLE DAMS.

History.—While the bear-trap was earlier in use on the Lehigh River, yet the pioneer of movable dams on navigable rivers was the needle dam, invented by M. Poirée in 1834, and first constructed at Basseville, France. It is called a needle dam because the wall which holds and supports the water is made of needles or wooden spars ranged side by side across the river.

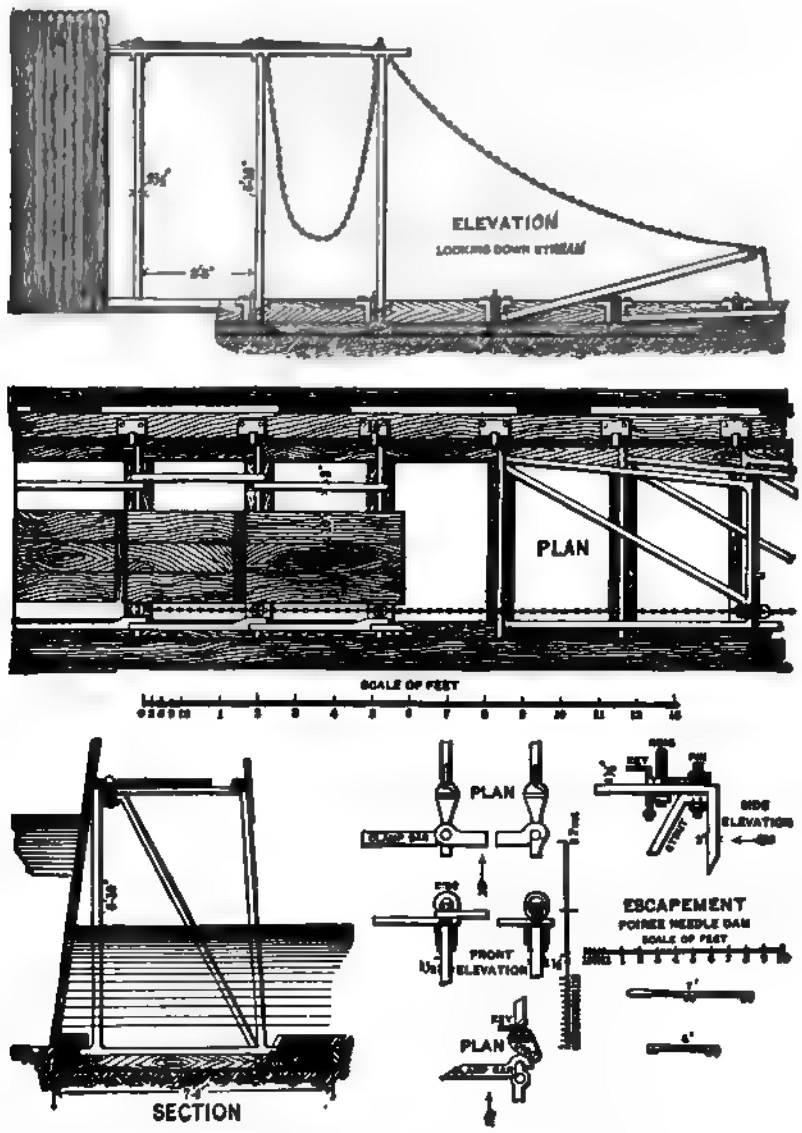
Modern needle dams are constructed throughout on almost the same principles as this first dam. The curtain of the dam is formed of the needles, which are of a size suited to the head of water, yet rarely larger than one man can control. Their bottoms rest against a sill, their tops against bars known as escape-bars, which are in turn supported by trestles turning on hinges on the floor and lowered behind the sill when not in use. A walkway, hinged to the trestles, or of loose planks, provides access for maneuvering when the dam is up.

In the Basseville dam the trestles were placed 3¼ feet (one meter) apart, and weighed 242 lbs., with a height of 6¼ feet, the lift being only 3¼ feet. The distance between the trestles is now made about 4 feet, only one example being known at present where it has been exceeded—that of the Louisa dam in the United States, where the trestles on the weir are 8 feet apart. There appears no reason, however, against using wider spans where proper appliances are provided for maneuvering, and as a precaution against drift this would be very desirable.

Maneuvers of Trestles.—All the trestles are connected, either by chains made in separate lengths and fastened to the heads of adjacent trestles, or by one long chain running through the heads, and fastened to them at the proper intervals by clamps or by latches, and worked by a winch on the masonry. The latter method is the one almost universally adopted in the modern needle dam, and by its means 2 to 6 trestles can be lifted at the same time. When it is desired to raise the dam the first trestle is raised up and connected to the masonry by the escape-bar and by the floor, and the same process is followed with the succeeding trestles until all are in place. Where a continuous chain is used it is released from each trestle in turn, as the latter becomes vertical, by a man on the foot-bridge; where separate chains are used a portable winch usually is carried from one trestle to the next. After all the framework is in place the needles are put in, sometimes from a boat, but usually from the foot-bridge.

To lower the dam, the reverse operation is pursued, the needles being first removed and the trestles lowered in order.

Placing and Removing Needles.—Small needles are usually placed and removed by hand. To place them, the head is held against the escape or support-bar, and the end plunged up stream in the current which draws it round till it strikes the sill. In



General Drawing of an Early Poirée (Needle) Dam.

the Marne dams, where the needles are 4¾ inches square and weigh up to 100 lbs., the same method is used; but the down-stream sides are provided with hooks which fit over round support-bars, thus holding the heads in the exact position desired. As the length from the hook to the end exceeds the length from the support-bar to the sill by ⅜ inch to ¾ inch, the foot of the needle scrapes along the floor just as the needle becomes upright. This takes away the shock from the support-bar almost completely, and assures the normal placing of the needle.

View of Down-stream Side of Pass and Weir, Looking towards the Abutment.

General View of the Lock and Dam.

NEEDLE DAM AT LOUISA, KY., U. S. A.

[a] (To face p. 219.)

The removal of these "hook needles" is usually effected by a crab carried on a truck, which raises the needle till it passes over the sill, when it swings on the support-bar and in the current. It is then lifted up by hand from the footway and loaded on to a car. A boat is only used in exceptional cases, and when the pools have reached the same level.

Another method of removal is that known as escaping. The escape-bar, which in all earlier dams was a loose bar connected when in use to the trestles by claw ends or by bolts, was later on hinged at one end so as to swing horizontally, the other end resting against a post shaped so that on being turned it released the end of the bar and allowed it to swing. The force of the water then carried the needles through the opening and they were picked up below. This is known as the Kummer escapement, from the name of its originator. The method has been largely adopted in Belgium and elsewhere, although the fixed bar is still employed in certain cases.

Another method, which has been in satisfactory use for some years and which does not bruise the needles, as often happens in the method of escapement, is to attach a long chain or rope to their up-stream sides and pull them away with a crab or an engine.

Dam at Louisa, Ky.—This dam, which was completed in 1896, on the Big Sandy River, in Kentucky, was the first of the needle type in the United States, and presents certain features which have proved to be an advance in some ways in the design and operation of needle dams. The original project called for a fixed dam, but before its completion strong opposition on the part of those connected with the river commerce developed, as it was feared that the placing a stationary dam in a river carrying so much sand would cause the pool to silt up, and impede rather than benefit navigation. The plan was accordingly changed to that of a movable dam, and owing to the small low-water discharge of the river (48 cubic feet per second), needles were adopted.

The pass is 130 feet long, with 13 feet of water on the sill and trestles 4 feet apart, and the weir is 140 feet long, with 7 feet of water on the sill and trestles 8 feet apart.

As the river carries large quantities of sand it was desirable to have the recess behind the sill as shallow as possible, to avoid the accumulation of deposit over the trestles. The latter are accordingly shaped like an inverted V without any axle, the bracing, etc., being placed so that they lie down, one inside the other, instead of one on top of the other, as is the usual way. By this means a height of sill of only 15 inches is needed. They are all raised by a continuous chain worked from the masonry, and can be lifted or lowered, if necessary, six at one time. The chain rolls on sprocket-wheels in the heads, and can be attached to or released from them at any point by means of latches.

The needles, both for the pass and for the weir, are made of white pine, 12 inches wide, and weigh apiece 263 lbs. and 80 lbs., respectively. The former are about 14 feet long, $8\frac{1}{2}$ inches thick at the bottom and $4\frac{1}{2}$ inches at the top, being designed for a

strain of 1200 lbs. per square inch; the latter are about 8 feet long, $3\frac{1}{2}$ inches thick at the bottom and $2\frac{1}{2}$ inches at the top. The width of 12 inches was adopted to save leakage, and has proved very satisfactory, as with an extreme low-water discharge of 48 cubic feet per second, and a head of 12 feet 2 inches on the pass, the total leakage through the dam was only 9 cubic feet per second.

Under a full load the needles show a considerable deflection, but do not take any permanent set, and the only cause of breakage has been the presence of knots or other defects in the timber. They are handled partly by men and partly by a small derrick-boat with an engine, and give little trouble in maneuvering. Those on the weir are placed by hand, and if any have to be put in under a full head, as in regulating, their tops are held against the escape-bar and the ends plunged in the water as before described. In this case a rope is generally placed over the lower ends and used as a check against the force of the water. The pass needles are placed by the derrick-boat, and can be put in without great difficulty with a head of 3 or 4 feet. If the dam is raised during the dry season a hook attached to a line running to the engine on the maneuvering boat is used to press the needles all together, thus closing any spaces.

The regulation is done principally by *repoussing*, or holding up stream about 12 inches the heads of alternate needles, thus allowing the water to escape between. Next to the pier, on the weir side, a space of 12 feet is left, provided with two rows of needles, one above the other, supported by a lower and an upper escape-bar, and this also provides a convenient means of regulation and of passing débris. Several needles 6 inches wide have been substituted for 12-inch ones on the weir to facilitate regulation, being more easily removed. Lastly, in good weather, the water is allowed to flow over the tops of the needles to a depth of 6 or 8 inches, along the entire crest.

A method of placing the pass needles has also been used which afforded good results, although the use of the derrick-boat eventually proved simpler. In it the needles were placed on hinged shelves about a foot above the water, one needle being placed every few feet against the sill to serve as a guide. The shelves were kept from turning by triggers attached to a line, and when all was ready the triggers were released, and the shelves revolved, allowing the needles to drop into the water all together.

The removal of the needles is effected by attaching the up-stream side of each one to a chain, allowing about 2 feet of slack between, and then pulling it from the boat, which is moored about 100 feet above the dam. By this means all the weir needles can be removed under a full head in a very short time, but the operation is always done slowly, so as to retain the pool at about the proper level. The trestles are also provided with swinging escape-bars, but as the needles become bruised against the trestles in falling, owing to their unusual size, this method of release has been rarely used.

On swift rises drift in large quantities comes down the river, which has once or twice threatened to cause damage to the trestles. To guard against this as far as possible a rudder boom several hundred feet long is kept afloat above the lock, stretching at a very flat angle across the river. This holds the drift until enough of the dam has been lowered to remove the danger.

Raising the Weir Trestles.

Placing the Pass Needles by Means of Revolving Shelves.

Weir Trestles Lying Down.

NEEDLE DAM AT LOUISA, KY., U. S. A.

(To face p. 220.)

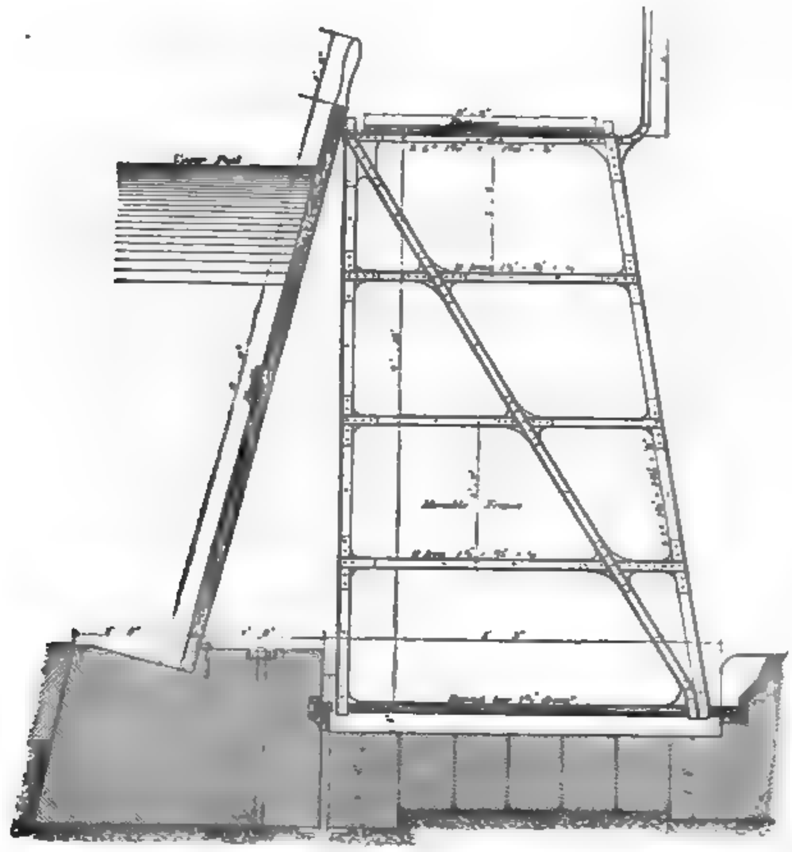
View of Pass Trestles and Needles.

NEEDLE DAM AT LOUISA. KY., U. S. A.

(To face p. 220.)

Needle Dams in Europe.—The most complete system of needle dams is to be found on the Meuse in Belgium, their construction having been completed about 1878.

The first dams on this river were closed entirely by needles, and in 1866 the system was continued by the construction of three Chanoine wicket dams. As these did not prove entirely suitable, the work was completed with nine dams using needles for the passes and wickets for the weirs. In these latest works the locks are 39 feet wide, with an available length of 328 feet, and a depth of 6.9 feet on the lower miter-sill, the upper and lower sills being on the same level. The passes are 150 feet in length,



SECTION OF NEEDLE DAM ON THE LOWER SEINE, PRIOR TO 1880.

with weirs 179 feet in length, and in some cases a fixed weir is provided also. The pass trestles are spaced about 4 feet apart, and weigh about 1100 lbs. apiece with all attachments, being raised by the method of separate chains and a portable winch. The needles, which support a head of 8.2 feet with about 10.2 feet on the sill, are 12.3 feet long and $3\frac{15}{16}$ inches wide, with a maximum thickness of $4\frac{7}{16}$ inches, and weighing about 55 lbs. They are supported by escape-bars on the Kummer system, and are removed by releasing the bar, allowing the needles to pass down stream.

The weirs are provided with wickets 4 feet 3 inches wide, and 7 feet 4 inches high, with a space of 4 inches between each, and are provided with butterfly valves in the upper parts, for the easier regulation of the pools. They are raised and maneuvered from a trestle bridge, and lowered by tripping-bars.

On the lower Seine was another system largely composed of needle dams, but these were replaced between 1878 and 1888 by dams of much higher lift, closed by gates or curtains, only three dams being now closed by needles. These support lifts of 9.1 feet, 9.3 feet, 10.5 feet.

On the Moldau, in Bohemia, a system is now under construction which includes needle dams, and which has been briefly described in another chapter.

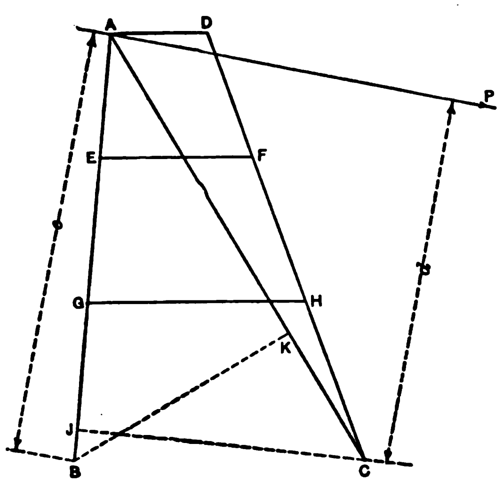


FIG. 14.

Other examples are to be found in France, Russia, and other countries.

Calculations for Trestles.—The trestle of a needle dam is practically a cantilever girder or truss, supporting a load at its end. It may thus be treated as an ordinary truss with tension and compression members and stiffening braces.

If $BADC$ (Fig. 14) represent a trestle supporting a load P from the needles, AB will be in tension and AC in compression, while ADC will support the footway or track and the other members, as EF and GH, bind the whole together. These last must be made strong enough to support blows from the drift, etc., and are usually made of the same section as AB.

To find the strains in AB and AC, taking moments about C we have

$$P \times d = AB \times CJ, \quad \text{or} \quad AB = \frac{P \times d}{CJ} \text{ (tension).}$$

Similarly for AC, taking moments about B, we have

$$P \times e = AC \times BK \quad \text{or} \quad AC = \frac{P \times e}{BK} \text{ (compression).}$$

Then if l = length of AC in feet and r = the radius of gyration of its section in inches, the ultimate strength per square inch may be found from the usual formula,

$$\frac{50000}{1 + \dfrac{(12l)^2}{24000 \times r^2}}.$$

Many needle trestles have been built with a second diagonal brace from B to D. In this case the strains will be divided between BA, AD, DC, AC, and BD, and may be found from equations of moments or by graphics. It is preferable, however, to use the simpler form of trestle, as it requires fewer pieces, and these, having in consequence to be made stronger, will resist shocks and wear better.

The omission of a continuous axle from B to C for all trestles except those of Boulé and Cameré dams, also appears desirable, as trestles have been in successful use for many years without axles, and the use of them complicates and adds to the ironwork.

In all ordinary cases, as in dams with trestles 4 to 8 feet apart, it will be found that a considerable excess of metal must be used above that required for the direct strains, in order to receive stiffness. Thus on the Louisa dam (Big Sandy River) the pass trestles are 4 feet apart, and each leg is composed of one 4-inch 8-lb. channel. The actual section needed for the direct strains is only about one-third of this.

On the weir of the same dam the trestles support a head of 7 feet, but were made of similar section, and are now used to carry a span of 8 feet, the original span having been 4 feet.

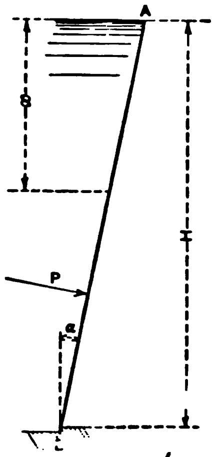


FIG. 15.

On the older needle dams of the Seine and of the Saône the stresses in the main members varied from 2000 lbs. to 2700 lbs. per square inch.

The width of base is made from six- to eight-tenths of the height.

Needles.—Suppose it is desired to design a needle to support a head of water H, the lower pool being left out of the calculation (Fig. 15). Let $H =$ depth of water on the sill in feet; $P =$ pressure of water on the needle; $w =$ width of the needle in feet; $t =$ thickness in inches required at the point of maximum moment. The length of the needle is then $H \sec \alpha$.

Then $P = H \sec \alpha \times w \times \frac{H}{2} \times 62\frac{1}{2}$ lbs. $= \frac{wH^2 \sec \alpha}{2} \times 62\frac{1}{2}$ lbs., of which one-third goes to A.

The bending moment M, at any point vertically distant x from the surface, is in inch-pounds.

$$M = \left(wH.\sec\alpha\,\frac{H}{2}\times 62\tfrac{1}{2}\text{ lbs.}\times\frac{x\sec\alpha}{3} - wx\sec\alpha\cdot\frac{x}{2}\cdot 62\tfrac{1}{2}\text{ lbs.}\cdot\frac{x\sec\alpha}{3}\right)\times 12$$

$$= \frac{wx\sec^2\alpha\,.\,62\frac{1}{2}\text{ lbs.}}{6}(H^2 - x^2)\times 12.$$

This moment is a maximum at a point vertically distant $\frac{H}{\sqrt{3}}$ from the surface of the water, in any beam supporting water level with its top and on one side only. Putting $x = \frac{H}{\sqrt{3}}$, we find

$$M = \frac{wH^3\sec^2\alpha \times 62\frac{1}{2}\text{ lbs.}\times 12}{9\sqrt{3}}\text{ inch-pounds.}$$

From the formulas for beams we have

$$M = \frac{SI}{c} \quad\text{and}\quad I = \frac{wt^3}{12},$$

where S = extreme fiber stress per square inch,
c = distance of center of gravity of section from outside in inches,
I = moment of inertia of section,
M, w, and t being as before stated.

Now $c = \frac{t}{2}$, and S must be assumed as desired, say 1000 lbs. per square inch, or more, according to the timber to be used. Combining the equations for I and M, we find

$$M = \frac{1000 \times wt^3}{12} \times \frac{2}{t} = \frac{1000 \times wt^2}{6}, \quad \text{or} \quad t^2 = \frac{6M}{1000 \times w},$$

from which t can be found.

For a head of water 18 feet, with the needle slightly inclined, t was found to be $11\frac{7}{8}$ inches, w being 1 foot, and S about 1000 lbs.

The usual form of needles adopted in European practice is of square section, varying from $1\frac{1}{2}$ inches by $1\frac{1}{2}$ inches on the first dams, with a length of $8\frac{1}{4}$ feet and a weight of 5 lbs., to $4\frac{3}{4}$ inches by $4\frac{3}{4}$ inches, on the more recent ones, with a length of $16\frac{1}{2}$ feet and a weight of 104 lbs. Small dimensions have been a desideratum with European designers, as the operation of the dams has been by hand, and it was necessary to limit the size of the needles to such as could be controlled by one man.

The lap on the sill is made from 4 to 6 inches.

Where there are appliances for handling them, needles of wide face are much preferable to narrow ones, as there are fewer joints and less tendency to warping, thus securing a minimum of leakage. With narrow needles, such as are generally employed, it is necessary in a dry season to use weeds, straw, canvas, or other means, to close the spaces between them. In any case, the thickness of the needle must not be greater than its width, or the pieces will turn in the current when being placed.

As regards the kind of timber, white pine appears to be the most satisfactory, as it possesses the greatest strength, weight for weight, and does not splinter easily. Oregon pine would probably make an excellent needle also. Georgia pine, while possessing less tendency to become water-soaked, splinters easily, and the section required to support a given head of water will weigh more than the section which would be required in white pine. Oak is not suitable, as it becomes water-soaked and is very liable to warp, the last disadvantage belonging to yellow poplar also, unless sawed from large trees. On the recently completed dams on the Moldau and the Elbe, in Bohemia, larch-wood was employed, the largest needles being about $4\frac{1}{2}$ inches square and 13 feet long, and weighing when wet 72 lbs. On other dams in Europe Riga fir has been generally used.

The section used for needles varies somewhat; some being square, some rectangular, while another form has a uniform width, but is larger at the point of greatest resistance than at either end. Hexagonal and semi-hexagonal needles, and needles with rubber up-stream facings overlapping their neighbors, have been proposed and experimented with, but none of these have come into general use. A hollow needle made of four planks nailed together and banded with iron has also been proposed.

General Remarks on Needle Dams.—The needle dam offers many advantages for rivers of small low-water discharge, as it can be designed to permit very little leakage, and is inexpensive in construction and operation. For wide rivers, however, espe-

General View of the Needle Dam at Klecan, Bohemia, taken from the Lock-wall, Showing the Method of Operating the Trestles by a Portable Winch

(This method has since been replaced by one long chain, operated from the pier.) The dam consists of two weirs each 127 feet long and one pass 131 feet long. The trestles are spaced 4 feet 1 inch apart. The work was completed in 1899.

(To face p. 224.)

cially where the rises are rapid, requiring much regulation, they are less adapted, owing to the number of pieces to be handled and cared for, and it is probable that in the United States their use will be confined to the smaller streams. They have not yet been developed to high lifts, since engineers have retained the view that each needle should be controllable by one man, but the example of the Louisa dam has shown that this is not a desideratum, and other examples are now under construction in which the size of needles and depth of water will be considerably increased.

DIMENSIONS OF TRESTLES OF NEEDLE DAMS.

Location.	Height.	Width of Base.	Ratio of Base to Height.	Distance, C. to C.	Lift.	Weight Each, Lbs.	Remarks.
Original Poirée trestles, about 1834..........	6′ 3″ to 6′ 6″	4′ 11″	$\frac{8}{10}$	3′ 3″	3′ 3″ to 3′ 6″	220 to 310	Frames of bar iron, $1\frac{3}{16}$″ sq.
Meuse Ardennaise.....	8′ 0″	5′ 3″	$\frac{66}{100}$	....	5′ 10″	300	
Martot (Lower Seine)..	11′ 0″	8′ 2″	$\frac{74}{100}$	....	9′ 10″ on sill	470	
Belgian Meuse.........	13′ 2″	8′ 4″	$\frac{63}{100}$	3′ 11″	8′ 3″	800	Footway 44″ wide, 1′ 8″ above pool.
Louisa, Ky., U. S. A. (Big Sandy River, 1896), pass.........	15′ 2″	9′ 10″	$\frac{65}{100}$	4′ 0″	13′ 0″ on sill	1150 complete	Footway 38″ wide, 1′ 6″ above pool. Weights include attachments.
Louisa, Ky., U. S. A. (Big Sandy River, 1896), weir..........	9′ 8″	6′ 5″	$\frac{66}{100}$	8′ 0″	7′ 0″ on sill	920 complete	Footway 38″ wide, 1′ 6″ above pool. Weights include attachments.
Klecan, Moldau (Bohemia, 1900)..........	13′ 4″	8′ 3″	$\frac{61}{100}$	4′ $1\frac{1}{2}$″	abt. 13′ on sill	....	Footway 30″ wide, about 1′ 4″ above pool.

DIMENSIONS OF NEEDLES.

Location.	Width, Inches.	Thickness, Inches.	Length.	Lift.	Weight, Lbs.	Remarks.
Original Poirée needles, about 1834..............	$1\frac{1}{2}$	$1\frac{1}{2}$	8′ $2\frac{1}{2}$″	3′ 3″	5	
Meuse, passes............	$3\frac{15}{16}$	Max. $4\frac{3}{16}$ Min. $3\frac{5}{8}$	12′ 4″	8′ 3″	55	Extreme fiber stress, 1060 lbs. per sq. in.
Lower Seine, old dams....	$3\frac{1}{8}$	$3\frac{1}{8}$	13′ 2″	7′ 0″	....	Extreme fiber stress, 2300 lbs. per sq. in. Too great in practice.
Joinville, Marne, 1885.....	$7\frac{7}{8}$	$4\frac{3}{8}$	13′ 9″	9′ 10″ on sill	108	Replaced later by needles $4\frac{3}{4}$″ sq.
Marne, 1897.............	4 to $4\frac{3}{4}$	4 to $4\frac{3}{4}$	16′ 5″	....	104	
Big Sandy River (Louisa, Ky., 1896), pass........	12	Bott., $8\frac{1}{2}$ Top, $4\frac{1}{2}$	13′ 10″	13′ 0″ on sill	263	Extreme fiber stress, 1230 lbs. per sq. in.
Big Sandy River (Louisa, Ky., 1896), weir........	12	Bott., $3\frac{1}{2}$ Top, $2\frac{1}{2}$	7′ 8″	7′ 0″ on sill	80	Extreme fiber stress, 990 lbs. per sq. in.
Klecan, Moldau (Bohemia, 1900)	$3\frac{3}{4}$ to $4\frac{3}{4}$	$3\frac{3}{4}$ to $4\frac{3}{4}$	10′ 9″ to 13′ 0″	10′ 0″	46 to 72	

COST OF NEEDLE DAMS.

Location.	Lift.	Cost per Foot Run. Fixed Parts	Movable Parts.	Total.	Remarks.
Meuse Ardennaise	5′ 11″	....	....	$97.50	Average of whole dam.
Belgian Meuse	8′ 3″	$110.00	$43.00	153.00	Cost of passes.
Saône	7′ 6″	....	....	110.00	Cost of weirs.
Martot, Lower Seine	9′ 10″	....	....	244.00	Cost of pass.
Louisa, Ky., Big Sandy River	13′ 0″ on sill	226.50	19.20	245.70	Average of pass and weir.

CHAPTER VII.

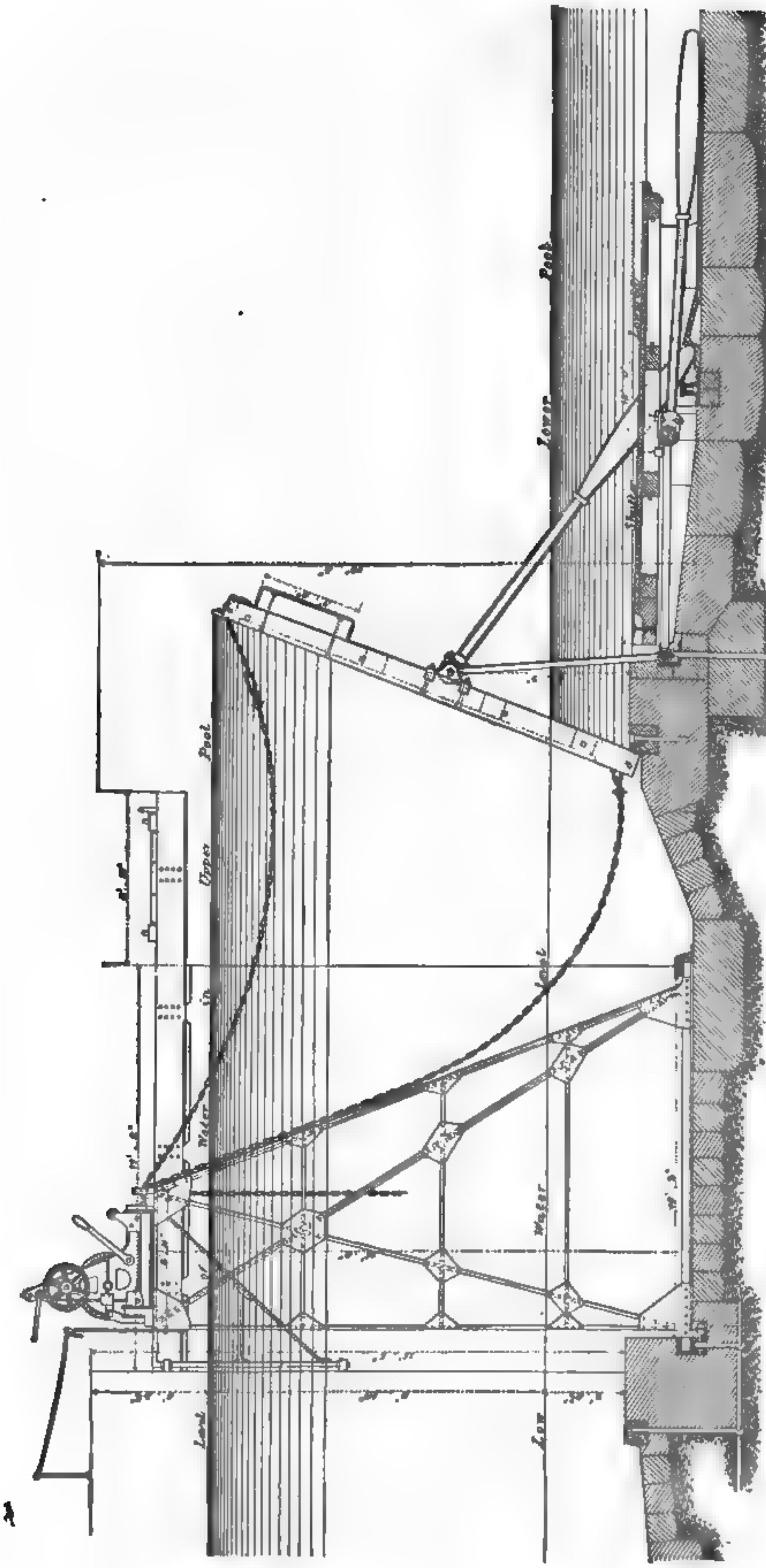
CHANOINE WICKET DAMS.

History.—As the Poirée dam was developed from the old-time plank dams, so the Chanoine wicket dam is the evolution of another type. Gates with horizontal axes fixed in the abutments had long been in use, as, for example, the valves used in the dikes of Holland, while the weirs of certain dams on the Orb (1778) were formed of shutters supported by hinges fastened to the floor Later on M. Thénard added the prop, the hurter, and the tripping-bar, and the invention of M. Chanoine in 1852 raised the axis of rotation to near the middle of the wicket, creating a type which has remained almost unchanged.

Description.—The Chanoine wicket is a shutter hinged near its middle, supported when up by a prop, and kept in position when revolving by a hinged frame known as the horse. The lower end of the prop rests against a casting on the floor called the hurter. When this end is moved from the support, the pressure of the water pushes down the wicket, which turns on the horse until it lies flat behind the sill, with the hurter, prop, and horse underneath. The part of the wicket above the axis of rotation is called the head, or chase, and the part below, the breech or butt.

In the heads of the wickets are sometimes placed other small wickets, known as butterfly-valves, turning on an axis framed into the main wicket and kept closed by latches. Their object is to afford a means of regulating the pool without having to operate the large wickets, and they are opened or closed by means of a hook-pole with which the dam-tender frees or closes the latches. They are not used in America, as drift would prove troublesome, but they are in use on the Meuse, at La Mulatière dam, and elsewhere.

The original idea of Chanoine was to place the axis at a point above the sill one-third of the length of the wicket, this being the center of pressure of the water. As the pool rose, this center would be changed to above the axis, and the wicket would swing automatically, reducing the pool to its proper level, when the wicket would swing back. All the early dams were constructed on this plan, being raised from boats and lowered by tripping-bars but it was soon found that the wickets were too sensitive, and while opening without trouble, they would not swing back without assistance, or until the pool had fallen considerably. Moreover, when one wicket opened it caused the lower pool to rise, creating more water below the dam and raising the center of pressure on the down-stream side of the other wickets. This caused others to open,



Section of Pass, Port-à-l'Anglais Dam, on the Seine (1870).
(Lowered by a tripping-bar.)

View of the Abutment and Part of the Weir of a Chanoine Wicket Dam (Kanawha River, W. Va., 1897).

Three wickets are standing in position next the abutment: one is on the swing, held by the chain; the others are lowered behind the sill. The wickets shown rise 8 feet 6 inches vertically above the sill and are 3 feet 9 inches wide; the trestles are 12 feet high and 8 feet apart. The chains for maneuvering the wickets and trestles were yet to be placed.

(To face p. 329.)

until the whole dam was on the swing and the pool level greatly lowered. For these reasons the axis was raised in all the later dams, as mentioned further on. Fixed, and in some cases sliding, weights or counterpoises were attached to the wickets, to regulate better the tendency to swing, but they did not prove very satisfactory. With a view to minimize the evil, experiments were tried on the upper Seine dams about 1865, when several of the wickets on the weirs were provided with chains fastening the butts to the sills, and were made wider at the same time in the heads so that they would swing before the others. The chains were of such a length that when the pool rose and swung the wickets, the latter could not pass an angle of forty-five degrees with the vertical, and would therefore right themselves much sooner than when swinging free. This method was in part successful, but it was soon found to give rise to another objection, which was that the wickets by being chained reduced the full spillway of the dam. In order, therefore, to pass rises which formerly could be disposed of by putting the wickets on a full swing, it became necessary to lower several, or to maneuver the heavy wickets of the pass. The operation of raising them again, which had to be done from a boat exposed to violent currents, was found to be difficult and dangerous. Finally, as the only solution of the problem, De Lagréné proposed placing a bridge of trestles above the dam from which the wickets could be maneuvered, and such a bridge has now come to be regarded as almost indispensable for proper operation.

Dimensions, etc.—A width of 3 to 4 feet has usually been adopted for wickets, the latter having been used on all American dams. From this is deducted a space of 3 to 4 inches for clearance between each one, this being necessary to allow for warping. The width of the pass and of the weir wickets is generally the same, although in a few cases the former is less, owing to the greater height. On the upper Seine, for example, certain of the passes have wickets 11.8 feet high and 3.3 feet wide, while those of the weir are 6.5 feet high and 4.3 feet wide. The angle of inclination to the vertical, where Pasqueau hurters are used, is about twenty degrees; but on the earlier dams, where the tripping-bar was relied on for lowering, the angle was much less. The lap on the sill is 4 or 5 inches.

The wickets are made of wood, framed and bolted. The only example of iron wickets is to be found at La Mulatière dam near Lyons, although others will shortly be in existence. An iron wicket can be designed so that its skin-plate only, and not its uprights, will overlap the sill, thus reducing one cause of leakage which is inevitable with the wooden wicket.

Hurters.—The hurter consists essentially of a shoulder and of two grooves, one of which guides the end of the prop when the wicket is being raised till it falls against the shoulder, while the other guides it back when released and leads it into position again ready for raising. Until 1879 all lowering had to be done by a tripping-bar, or by pulling the props sideways till they cleared the shoulder. This was facilitated by sloping the latter slightly in the horizontal plane, so the prop would bear against a slanting surface. The safe maximum of this angle, as found by experiments made some years ago at the dam of Conflans in France, is three degrees.

In the year mentioned, however, M. Pasqueau introduced the double-stepped hurter, which has practically superseded the former kind, and which provides a second shoulder up stream of the first one. In this case the entire wicket is pulled up stream, and the end of the prop leaves its support and falls over the second shoulder into a groove. The wicket is then released and the prop slides along the groove which guides it back into position ready for raising.

The various types of hurters are shown on the corresponding drawing.

Tripping-bar.—This is a bar provided with teeth and moving along just under the feet of the props. It is worked by gearing in the pier or the abutment masonry, the teeth being arranged so as to strike each prop in turn and pull it sideways from the shoulder, thus allowing the wicket to fall. A cushion of water is always necessary, as the falling parts will otherwise be damaged or broken by the shock.

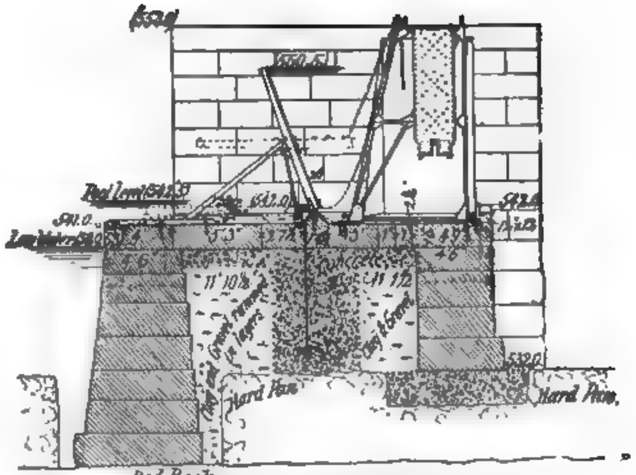
As the total travel of the bar is limited to the distance between any two props, minus the width of the tooth, this arrangement is inapplicable to wide passes, since many props would have to be thrown at once in order to get all of them down within the limit of the travel. About 160 feet is the widest opening to which it can be applied, and it has then to be divided into two parts, one working each way. At first only one tooth engages, but at the last several teeth have to be pulling together.

The tripping-bar has been in successful use on the Meuse, the Upper Seine, and elsewhere for many years. Some trouble was at first experienced with its use, owing to insufficient power and to the obstruction of gravel, but in the later designs these objections were overcome, the machinery being given a force of 12 to 15 tons, and pulling the teeth through any ordinary obstacle.

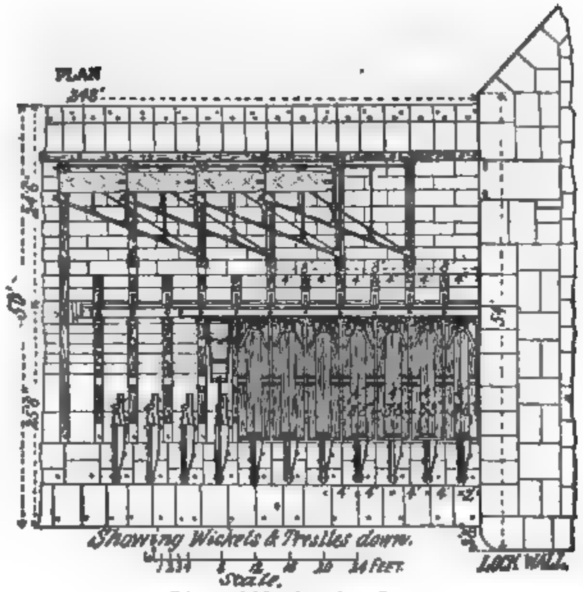
It has not found favor in America, partly owing to the width of the rivers to which Chanoine dams have been applied, and partly to the fact that on the dams where used the bar was weak and was also raised off the floor, as in the French design, so that drift caught under it frequently and unseated it. Its use was therefore abandoned, but the few times in which it was maneuvered showed it to be a valuable device, as the wickets could all be lowered from the shore and with ice against them. To guard against danger from drift, etc., the bar should be made of flat section, say 5 inches wide and ¾ of an inch thick, the teeth being riveted on as bent plates, and it should be recessed so that its top will not project above the masonry. It may slide on rollers or on surfaces provided on the hurters, the latter plan being more reliable, as the rollers may become clogged or broken. The machinery, which should possess a large excess of power, must be arranged to act in either direction, since it has to push the bar back to place after the wickets have been thrown. Guides must be provided every few feet to prevent the bar from lifting up; these can be bolted to the hurters.

Where it is desired to use a Pasqueau hurter, which would always be advisable since the bar may at some time become unworkable, the latter may be arranged to slide between the two shoulders, the prop passing over it when the wicket is maneuvered.

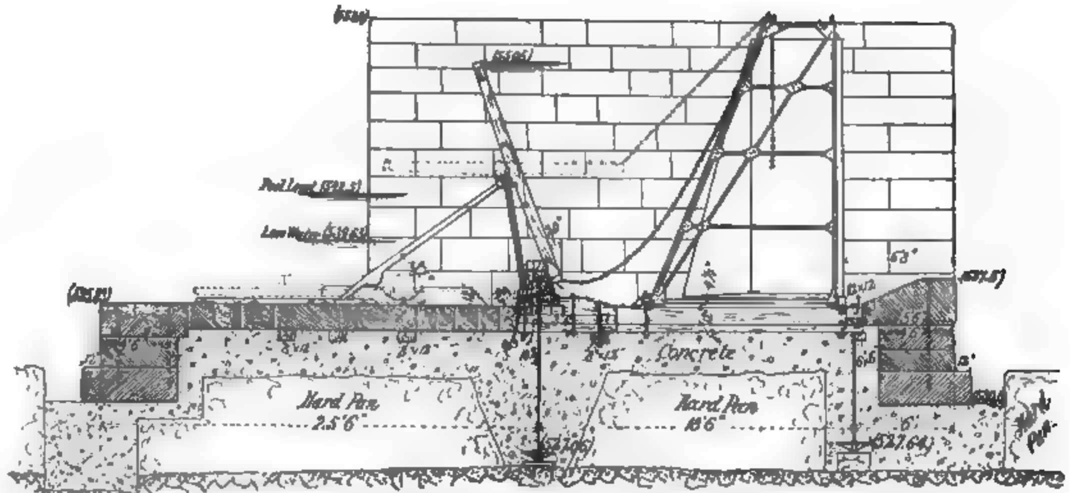
Maneuvers.—A wicket dam is usually raised from a service bridge placed on its



General Section of Weir—Abutment in Elevation.



Plan of Navigation Pass.



General Section of Navigation Pass—Center Pier in Elevation.

SECTIONS, ETC., OF PASS AND WEIR, DAM NO. 7, KANAWHA RIVER, W. VA.

(To face p. 232.)

up-stream side, and consisting of movable trestles, provided with floors or "aprons" and a service track on which the operating winch rolls. The trestles are placed 8 to 9 feet apart, and have their footway 2 to 3 feet above the pool. To raise the dam, the trestles are first pulled up by the winch, using the short chains attaching each trestle-head to the end of the apron of the next one, and setting the aprons and track rails in position as the maneuvers progress. The wickets are next pulled up by chains, which connect the bottom of each wicket with the head of the nearest trestle, and when the props are seated they are left "on the swing," that is, balanced on the horses in a horizontal position. In this way the pass and weir wickets can all be raised without obstructing the flow of the river to any appreciable extent.

When all is ready, the wickets are righted and the current catches the butts and forces them against the sill, thus closing the dam. This righting is done by pushing the butts down with a pole, which has usually to be operated by the winch, if the wickets are of any size. Another method, which is used on certain dams, consists in one or two men walking across the wickets and overbalancing each until it strikes the current, when they jump on to the next one and repeat the maneuver until all are righted. This is, however, attended with much danger, as if the men fall into the river they will almost certainly be caught in the horses and be drowned.

To regulate the pool, a few wickets with the corresponding trestles are lowered, or they are put on the swing and kept in that position by holding the breech-chains in the "stops" or chain sockets on the trestle-heads. When the discharge becomes low the spaces between the wickets are closed by means of square timbers, pushed down into them cornerwise.

To lower the dam, the props are displaced by the tripping-bar, or, if Pasqueau hurters are employed, the entire wicket is pulled up stream by the winch or from a boat, until the prop falls over the shoulder into the groove and the wicket is then lowered. On the weir this is done by pulling on the breech-chains; but on the pass, where the lowering of the weir has reduced the pressure, the head of the wicket is pulled up stream by a special grab-hook until the prop is free, when the hook is jerked off and the water pushes the wicket down. This saves having first to put the pass wickets on the swing, and permits of very rapid maneuvers. The breech-chains are then fastened in the stops by pins and the trestles lowered.

In dams dependent upon the tripping-bar, when the latter gets out of order, the head of each wicket is pulled up stream from a boat till its prop is free, and the latter is then pushed off the shoulder with a boat-hook from a skiff below and the wicket lowered.

La Mulatière Dam.—This dam, completed in 1879, near Lyons, France, at the junction of the Saône and Rhone rivers, is one of the most advanced examples of a Chanoine wicket dam. It was constructed under the direction of M. Pasqueau, and it was there that he first introduced the double-stepped hurter, which has since been widely adopted.

The pass of the dam is 340 feet in length, and is closed by wickets made of iron, 4.6 feet wide and 14.25 feet long, each being provided with a flutter-valve about 2.9 feet

wide and 5.1 feet long, operated by a hook pole in the usual manner. The sheathing-plates are $\frac{3}{16}$ of an inch thick. The wickets are operated by a steam-engine, which moves on a service bridge of trestles, the track being 6½ feet above pool level. These trestles are 9.8 feet apart and 22.3 feet high, and provided the first example where no long bottom axle was used, pins being employed instead. By this arrangement the depth of the recess behind the sill was reduced from 4 feet to 28 inches.

It was here also that steam-power was first employed instead of the old methods of hand-power, and the success of the work generally has had a considerable influence on wicket dams built since its completion.

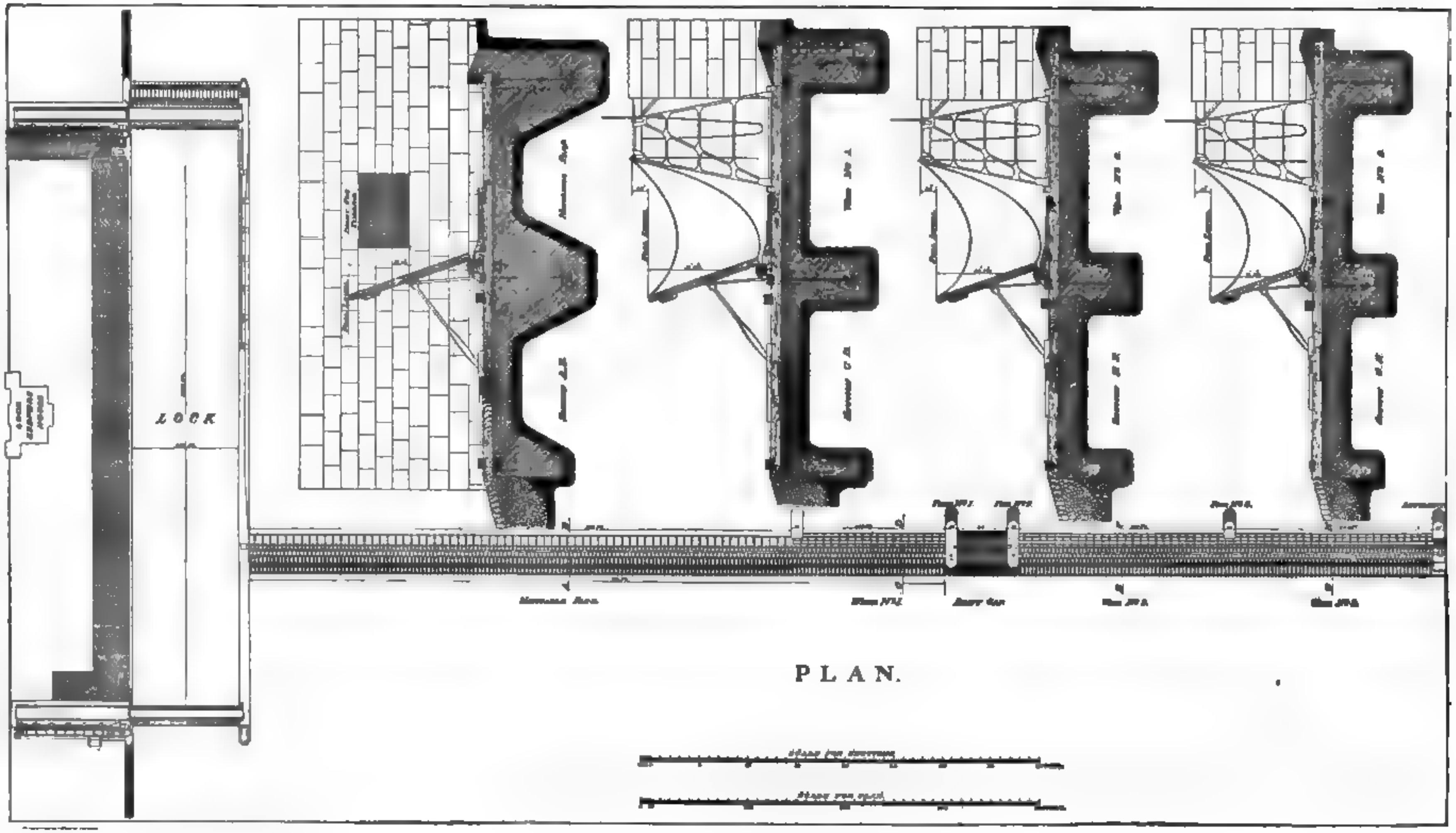
American Dams.—At present only one river in America, the Kanawha River, in West Virginia, has a completed system of wicket dams. They are eight in number, with lifts from 6¼ to 8½ feet, the weirs being from 210 feet to 364 feet in width, and the passes from 248 feet to 304 feet. The system was built between 1880 and 1898, and in connection with two fixed dams affords a depth of 6 feet over about 90 miles of river.

These dams are generally similar in design to those of Europe, and are operated with trestle bridges and winches as before described. The locks are 55 feet wide and 342 feet between hollow quoins.

On the Ohio River a system of locks and wicket dams has been commenced. The first of these, that of Davis Island, just below Pittsburg, is the only one in operation, although others will soon be completed. The dam consists of a navigation pass, 719 feet wide, and two weirs, affording a total opening of 1220 feet, which is closed by 305 wickets, spaced 4 feet apart and provided with Pasqueau hurters. The pass wickets are maneuvered from a boat with a steam-engine, the hoisting-line being attached to a pole, with a hook at its end, which the operator catches in the butt of the wicket. To lower the dam, the head of the wicket is pulled up stream until the prop clears the shoulder, when it is released and falls with the current. One of the weirs is maneuvered from a service bridge and the other by boat. The dam has also a bear-trap drift chute, 52 feet wide, located between the pass and the first weir, and a fixed dam separated from the second weir by an island. The lock is 110 feet wide and 600 feet long between the gates, the lift, when the pool below is at normal stage, being 6.1 feet.

Calculations—*Trestles.*—The trestles for wicket dams differ from those required for other types in that the only direct load they sustain comes from the pull of raising or lowering the wickets, and from the weight of the crab and its car. The indirect loads from drift, etc., are of course the same as elsewhere.

The direct load is a variable quantity, and can only be approximated. Its maximum occurs when the wickets have to be pulled out of deposit or drift which may have settled on them, and when the deposit is of any depth the winch is sometimes unable to move them. The load should therefore be assumed as not less than the maximum capacity of the winch, and it will usually be found sufficient to take it as equal to two-thirds of the total water pressure on the wicket when standing, multiplied by the angle between the chain and this pressure. The angle may be taken as



GENERAL PLAN AND SECTION OF THE DAVIS ISLAND DAM, OHIO RIVER, NEAR PITTSBURG, PA.

(To face p. 233.)

the same when the wicket is upright as when it is lowered, the actual difference being small.

Thus in Fig. 16, according to this supposition, the direct load on the chain will be $\frac{2}{3}P \cdot \sec\beta$, where P = the total water pressure on the wicket, and β = the angle between the chain and the direction of P. The water should be assumed at pool level, or above, with no water below, so as to provide for the most unfavorable condition. This load is

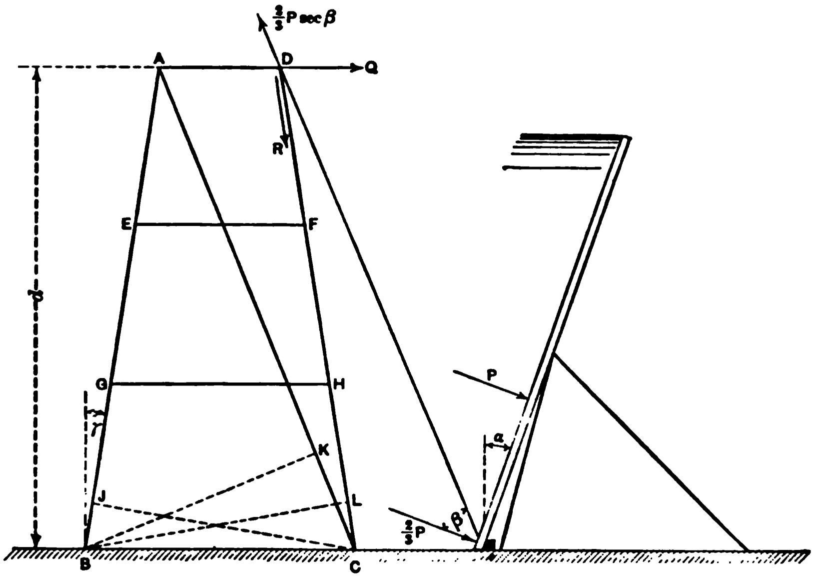


FIG. 16.

usually a maximum on the weir of a dam, as its wickets are lowered first, and by the time the pass is reached the difference between the upper and lower pools has been much reduced. The crab, however, should have considerably more than the theoretical power required, and the trestle should be designed accordingly, since the wickets have sometimes to be pulled out of deposits of sand or mud.

To find the strains in the trestle, assume the line of pull to pass through D and resolve it there into two components, Q and R, along AD and DC.

Taking moments about C, we have

$$Q \times d = AB \times CJ, \quad \text{or} \quad AB = \frac{Q \times d}{CJ} \text{ (tension).}$$

Taking moments about B, we have

$$Q \times d + R \times BL = AC \times BK, \quad \text{or} \quad AC = \frac{Q \times d + R \times BL}{BK} \text{ (compression).}$$

The upward pull on the anchorage at B where γ = angle of inclination on AB = strain in $AB \times \cos\gamma$.

As mentioned in the calculations for needle trestles, it will be found that a considerable excess of strength must be supplied for stiffness, and experience can prove the only guide in such matters. On the Meuse dams, with a head of about $7\frac{1}{2}$ feet, the

trestles were made in the form of a St. Andrew's cross, of bar iron, about 1½ inches by 2 inches. On the Saône, on the pass of the Ile Barbe dam, the trestles were about 13 feet high, and made of channel-irons about 2½ inches wide. On the Kanawha River the pass and weir trestles are 16 feet 9 inches and 11 feet 9 inches high respectively. They are built in the shape shown in Fig. 16, *DC* on the pass being composed of two 4-inch 9-lb. channels and all the other members of one channel of similar size. On the weir, *DC* is formed of a 3-inch 9-lb. I beam, and the other members are of a single 3-inch 6-lb. channel. The trestles are all 8 feet apart.

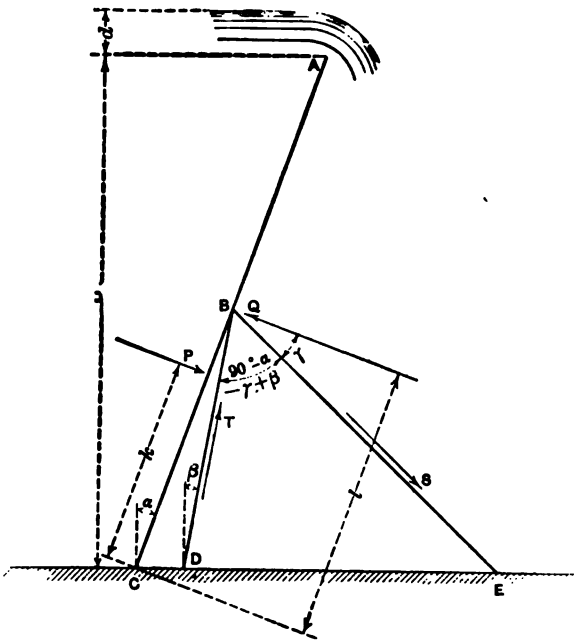


FIG. 17.

The trestles of La Mulatière dam, near Lyons, are 9.8 feet apart, in the form of a St. Andrew's cross, and carry a steam-engine with which the dam is operated. Each member consists of a single channel-iron, 5½ inches wide. Their height is a little over 22 feet, and the width of base 11 feet 6 inches.

Wickets—Strain on Horse and Prop.—To find the maximum strains on a wicket of the Chanoine type we will assume the water to be flowing over the top to a depth *d*, and that there is no water below. In practice *d* may vary from 0 to 1½ feet (see Fig. 17).

Let H = depth of water on sill in feet;

d = depth of overflow in feet;

w = width of wicket in feet;

P = total of pressure of water;

Q = that part of P acting at B;

S = strain in prop;

T = " " horse;

α, β, γ = angles of inclination as shown;

k = distance from base to center of pressure;

l = " " " " " " support.

Then $$P = w \times H \sec\alpha \times 62\tfrac{1}{2} \text{ lbs.} \times \left(\frac{H}{2} + d\right).$$

The distance k may be found by considering P as composed of a rectangle and a triangle of pressures (Fig. 18), thus:

$$P = P' + P'' = dwH \sec\alpha \times 62\tfrac{1}{2} \text{ lbs.} + wH \sec\alpha \cdot \frac{H}{2} \times 62\tfrac{1}{2} \text{ lbs.}$$

General View of the Navigation Pass (600 feet long), Dam No. 6, Ohio River, 1898, Showing the Chanoine Wickets Raised in Position

(Work just completed and yet in coffer-dam.)

(To face p. 235.)

Then, taking moments about C, the moment of P = sum of moments of $P' + P''$, or

$$P.k = P' \times \frac{H \sec \alpha}{2} + P'' \times \frac{H}{3} \sec \alpha = \frac{wH^2 \sec^2 \alpha}{2}\left(d + \frac{H}{3}\right) \times 62\tfrac{1}{2} \text{ lbs.}$$

Whence $k = \dfrac{wH^2 \sec^2 \alpha}{2P}\left(d + \dfrac{H}{3}\right) \times 62\tfrac{1}{2} \text{ lbs.} = \dfrac{H(H + 3d) \sec \alpha}{3(H + 2d)}.$

To find Q, take moments about C, whence

$$Q.l = P.k, \text{ or } Q = \frac{wH^2 \sec^2 \alpha}{2l}\left(d + \frac{H}{3}\right) \times 62\tfrac{1}{2} \text{ lbs.}$$

Then from the proportion between the sides and sines of a triangle,

$$S = \frac{Q \times \sin(90^\circ + \alpha - \beta)}{\sin(90^\circ - \alpha - \gamma + \beta)} = \frac{Q.\cos(\alpha - \beta)}{\sin(90^\circ - \alpha - \gamma + \beta)} \text{ and } T = \frac{Q.\sin \gamma}{\sin(90^\circ - \alpha - \gamma + \beta)}.$$

The pressure of the wicket against the sill at $D = P - Q$, and the upward pull on the anchorage at $D = T \cos \beta$.

In the preceding calculations we have neglected any water pressure below the wicket, in order to obtain the maximum strains which can occur. With the axis of rotation placed at the height dictated by experience and mentioned a little farther on, it has been found that wickets do not trip themselves under any ordinary conditions of practice, and that they will support without derangement a pool-level 12 to 18 inches above their tops. The latter condition is one which sometimes proves of much benefit, not only when an unexpected rise comes, but also where an extra channel depth is required for a short while to float too heavily loaded craft.

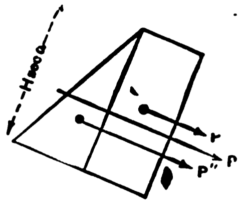


FIG. 18.

At the mouth of a stream, however, tributary to a large river which may cause backwater during a flood, the tributary itself being at an ordinary stage, it will be necessary to adjust the height of the prop so that the wicket will not trip before the backwater is level with the pool above. Unless this be provided against the wickets will begin to swing themselves too soon, thus reducing the upper pool-level and the depth of the water at the lock above and on the shoals. Such cases have occurred and have caused trouble to craft.

To find the height at which the prop should be placed in order to avoid this self-operation, let P, H, d, etc., represent the same quantities as before, and let h represent the depth of the water below, and R its pressure (Fig. 19).

Then $R = wh \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.} \times \dfrac{h}{2} = \dfrac{wh^2}{2} \sec \alpha \times 62\tfrac{1}{2} \text{ lbs.}$

Its moment about $C = R \times \dfrac{h \sec \alpha}{3} = \dfrac{wh^3 \sec^2 \alpha \times 62\tfrac{1}{2} \text{ lbs.}}{6}.$

For the wicket to be on the point of swinging, the resultant of P and R must pass through B. This resultant $Q' = P - R$, or

$$wH \sec\alpha \times 62\tfrac{1}{2}\text{ lbs.}\left(\frac{H}{2}+d\right) - wh \sec\alpha \times 62\tfrac{1}{2}\text{ lbs.} \times \frac{h}{2} = w \sec\alpha \times 62\tfrac{1}{2}\text{ lbs.}\left(\frac{H^2}{2}+dH-\frac{h^2}{2}\right).$$

Taking moments about C, we have

$$Q'.l' = P.\,k - R\cdot\frac{h \sec\alpha}{3}, \quad\text{or}\quad l' = \frac{3Pk - Rh\sec\alpha}{3Q'}.$$

Substituting the values of P, k, etc., as before found, we have

$$l' = \frac{\dfrac{wH^2\sec^2\alpha\,(3d+H)\times 62\tfrac{1}{2}\text{ lbs.}}{2} - \dfrac{wh^3\sec^2\alpha \times 62\tfrac{1}{2}\text{ lbs.}}{2}}{3w\sec\alpha \times 62\tfrac{1}{2}\text{ lbs.}\left(\dfrac{H^2}{2}+dH-\dfrac{h^2}{2}\right)} = \sec\alpha \times \frac{H^2(H+3d)-h^3}{3(H^2+2dH-h^2)}.$$

In order therefore to maintain the stability of the wicket the axis of rotation must be above the base a distance equal to or greater than l'.

In the earliest dams constructed the axis of rotation was placed above the base a little over one-third of the length AC. It was found, however, that the wickets were entirely too sensitive, and would swing with very little provocation, lowering the pools suddenly and causing a general disturbance of levels. In the dams of the upper Seine and other rivers, constructed between 1860 and 1870, the axis was accordingly placed at $\frac{42}{100}$ of the length above the sill; in the pass of Port-à-l'Anglais at Paris (1870) it was further raised to $\frac{47}{100}$ of this length; while at the dam of La Mulatière at Lyons (1879) it was placed at $\frac{50}{100}$ of the length. On the weirs of the Kanawha River in this country it is generally located at $\frac{42}{100}$, and on the passes at $\frac{47}{100}$ of the height above the sill. De Lagréné recommends that in navigable passes the axis be placed at one-half the height above the sill, and on weirs at a little more than one-third of the same distance.

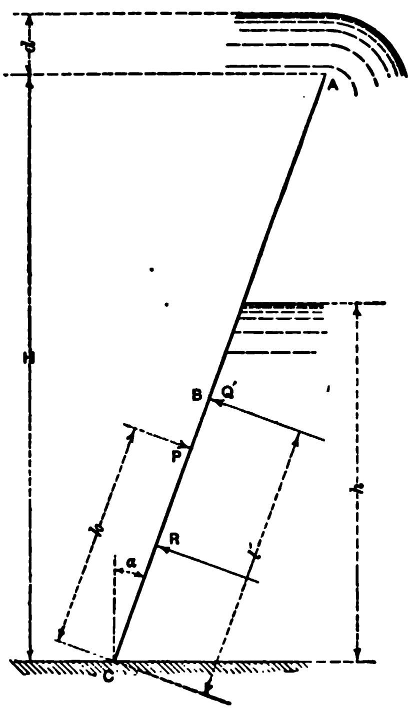


Fig. 19.

Horse.—On the earlier dams, where maneuvered by a tripping-bar, the horse BD was slightly inclined up stream, but when used with hurters of the Pasqueau type it must be inclined down stream, and so placed that it cannot be pulled up-stream far enough to pass the vertical and so throw the weight of the wicket on its up-stream side. A neglect of this precaution will sometimes cause trouble in lowering tne wickets.

VIEW SHOWING CHANOINE WICKETS, PROPS, ETC., OF THE NAVIGATION PASS, DAM NO. 6, OHIO RIVER, 1898.
(From sill to pool is 13 feet 2 inches.)

(To face p. 237.)

The journal-boxes should be bored $\frac{1}{32}''$ to $\frac{1}{16}''$ full, to secure easy movement and prevent rusting up.

The sectional area required in the horse for the tension T and to resist strains from the chain in raising is small compared with that required to stand drift, twisting, etc., and experience is consequently the best guide for proportioning it. The upper cross-piece, to which the head of the prop connects, must be made strong enough to support the bending moment caused by the latter, which equals $\frac{T}{2} \times \frac{f}{2}$, where f = width between the uprights of the horse. This piece is sometimes made with ears forged to and projecting above it, and provided with a pin to which the prop connects. This method, however, is objectionable, as it causes a twisting strain, besides adding to the cost of manufacture. Dimensions employed in actual practice will be found in the next paragraph.

Prop.—The strain S on the prop is essentially one of compression, and should be determined as for a "pin-bearing" column. A section must first be assumed, and if r = its radius of gyration in inches, and l the length of the prop in feet, the ultimate strength per square inch by the usual formula for columns will be in pounds.

$$\frac{50{,}000}{1 + \frac{(12l)^2}{18{,}000r^2}}.$$

As in the horse, the section should be made ample. Thus for the Port-à-l'Anglais dam (1870) the strain allowed on the props was only 1600 lbs. per square inch.

The head of the prop should be bored $\frac{1}{8}''$ to $\frac{3}{16}''$ full, to allow the proper lateral movement when sliding along the hurter.

The lower portions of the props for a pass are usually forged out to a considerable increase of area. The object of this is to provide a weight which the water cannot easily move, and so prevent a proper seating being obtained in the hurter. In view, however, of the fact that this increase has been found unnecessary on weirs under an 8-foot head, where the rush of water is much more violent than on passes, it would seem that the additional weight given to props on the latter is somewhat superfluous.

The following are examples of sizes used for horses and props:

SIZES OF PROPS.

Location.	Depth on Sill.	Diameter of Prop.	Length of Prop.
Pass of Port-à-l'Anglais, Paris	11′ 10″	$3\frac{1}{2}''$	11′ 10″
Passes of Upper Seine	9′ 10″	$3\frac{1}{2}''$	8′ 10″
Weirs of Belgian Meuse	About 7′ 5″	$3\frac{1}{2}''$	6′ 3″
Passes of Kanawha River, U. S. A.	13′ 0″	$3\frac{3}{8}''$	12′ 8″
Weirs of Kanawha River, U. S. A.	8′ 6″	3″	7′ 10″
Passes of Ohio River	13′ 2″	$3\frac{1}{2}''$	14′ 7″

SIZES OF HORSES.

Location.	Depth on Sill.	Braces.	Diameter of Journals of Top Arm.	Diameter of Journals of Lower Arm.	Uprights.
Pass of Port-à-l'Anglais, Paris, 1870	11′ 10″	Two, horizontal	Center, 5″ Ends, 2⅛″	Center, 3⅛″ Ends, 2⅛″	Bar iron, 3⅛″ × 1⅜″.
Passes of Upper Seine, prior to 1870	9′ 10″	Two, horizontal	Ends, 2⅜″	....	Bar iron, 2½″ × 1¼″.
Weirs of Belgian Meuse, about 1876	About 7′ 5″	None	Center, 2 11/16″ Ends, 2⅜″	Ends, 2⅜″	Bar iron, 2⅜″ × 1⅜″.
Kanawha River, U. S. A., 1896, pass	13′ 0″	Two, horizontal	Center, 5½″ Ends, 3″	Center, 2½″ Ends, 2½″	Two 3″ channels, back to back.
Kanawha River, U. S. A., 1896, weir	8′ 6″	One 1″ × 2″ diagonal	Center, 2 3/16″ Ends, 2″	Center, 2¼″ Ends, 1¾″	Bar iron, 2″ × 1½″.
Ohio River, 1899, passes	13′ 2″	Two 3″ × ¾″ diagonal	Center, 4⅞″ Ends, 2¾″	Center, 2⅞″ Ends, 2⅞″	Bar iron, 3″ × 1½″.

Frame of Wicket.—The uprights of the wicket AC (Fig. 20) are subject to a bending moment, the maximum of which occurs at the point of support at B, BA acting as a cantilever. BC is usually made of the same dimensions as required at B; hence, if the part AB is made strong enough to support the maximum moment, BC will possess sufficient strength also.

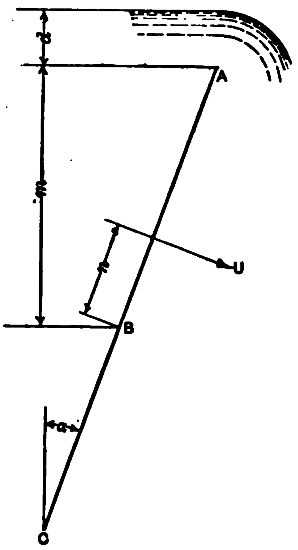


FIG. 20.

Let U represent the water pressure on AB, n the distance in feet of its center of pressure from B, m the vertical distance between A and B in feet, and the other letters as assumed in the preceding calculations. Then

$$U = wm \sec\alpha \times 62\tfrac{1}{2} \text{ lbs.} \times \left(\frac{m}{2} + d\right).$$

The distance n, which may be found as shown for the distance k on page 235, is

$$\frac{m\,(m + 3d) \sec\alpha}{3\,(m + 2d)}.$$

The total bending moment at B, in inch-pounds, is therefore

$$U \times n \times 12 = \frac{wm^2 \sec^2\alpha}{2} \times 62\tfrac{1}{2} \text{ lbs.} \times \left(d + \frac{m}{3}\right) \times 12.$$

The sections required can then be found, as shown in the calculations for needles, by the usual formulas,

$$S = \frac{Mc}{I} \quad \text{and} \quad I = \frac{bt^3}{12}.$$

General View of Chanoine Dam No. 11, Kanawha River, W. Va., U. S. A.

The view is taken from the abutment side, and the pass is seen next the lock, with the weir in the foreground. The water is about 15 inches above pool level. The weir is 364 feet in length and the pass 304 feet. The chamber of the lock has a width of 55 feet and an available length of 313 feet. The lock and dam were opened to navigation in 1898.

(To face p. 210)

The following are some examples of the sizes used in practice:

Location.	Depth on Sill.	Width of Wickets.	No. of Uprights.	Width of Uprights.	Thickness of Uprights.	Thickness of Planking.
Pass of Port-à-l'Anglais	11′ 10″	3.28′	2	12″	8″	2″
Kanawha River, passes	13′ 0″	3′ 8″	2	12″	9″	2″
Kanawha River, weirs	8′ 6″	3′ 9″	2	12″	6″	1½″
Ohio River, passes	13′ 2″	3′ 9″	2	12″	10″	2″

Remarks.—The Chanoine dam after some fifty years of trial has proved on the whole to be the type best adapted to large rivers subject to quickly rising floods, as it is not easily disabled, can be maneuvered rapidly, and does not possess many loose parts, such as accompany needle dams. The latter hold the water better, but on rivers of any size the low water-flow is usually more than enough to compensate for leakage.

One of the chief objections to the wicket dam is that it requires a very wide foundation, in order to provide a base for the trestle bridge. Where this can be dispensed with by operating from a boat the cost is much reduced. As to its practicability in all cases, however, experience is conflicting. On part of the Davis Island dam, on the Ohio River, a service bridge was originally provided, but it was destroyed one summer by drift, since which time the maneuvering of that portion has been done entirely from a boat. The wickets are 3 feet 9 inches wide and about 13 feet high.

On the Meuse, in Belgium, a boat is similarly used to raise the passes of the wicket dams, but service bridges were provided for all the weirs, where they were deemed necessary in order to control the wickets for the regulation of the pool.

On the upper Seine, after experiments in using a boat for the weir, it was deemed preferable to employ service bridges, a boat being found unsuited for night-work and for easy regulation, and they were accordingly added.

Where steam is employed there seems to be no objection to using a boat instead of a bridge, at least for a pass. With a weir, however, a bridge is desirable, although probably not necessary, as it allows an easier control of the wickets when they have to be swung to regulate the pool.

One objectionable feature of a service bridge is the floor sections, or aprons, as when the dam is being lowered, as well as when it is down, the current almost invariably works some of them free, so that their ends rise in the water and are then very liable to be injured by drift or by boats. This trouble is noticeable on any movable dam which has trestles provided with aprons, and it occurs chiefly on the weirs, since there the currents are more violent than in the passes. Many expedients have been tried to overcome it, but none has entirely succeeded. At La Mulatière dam the rails were braced together and hinged to the trestles, and a light floor of plank, of a total width of 4 feet, was fastened between them. By this means the surface exposed to the current was reduced and the weight considerably increased. This plan

appears to have several advantages over the usual construction, among which is the fact that it avoids the handling and transportation of the rails.

DIMENSIONS OF TRESTLES OF CHANOINE WICKET DAMS.

Location.	Height.	Width of Base.	Ratio of Base to Height.	Distance, C. to C.	Lift.	Weight, Each, lbs.	Remarks.
Belgian Meuse...........	8′ 2″	5′ 5″	$\frac{66}{100}$	4′ 8″	7′ 6″ on sill	....	Footway 44″ wide, 1′ 8″ above pool.
Port-à-l'Anglais, Paris, 1870	15′ 9″	10′ 2″	$\frac{64}{100}$	3′ 7″	9′ 10″	1325	Footway 33″ wide, 1′ 8″ above pool.
La Mulatière, Lyons, 1879..	22′ 0″	11′ 6″	$\frac{52}{100}$	9′ 10″	11′ 10″ (max.)	....	Footway [48″ wide, 6′ 6″ above pool.
Kanawha River, W. Va., Pass of No. 7, 1893......	16′ 10″	11′ 10″	$\frac{65}{100}$	8′ 0″	13′ 0″ on sill	about 1800	Footway 29″ wide, 2′ 6″ above pool.

COST OF CHANOINE WICKET DAMS WITH SERVICE BRIDGES.

Location.	Lift.	Cost per Foot Run.			Remarks.
		Fixed Parts.	Movable Parts.	Total.	
Marne..........................	6′ 6″	....	....	$244.00	Cost of passes.
Saône	7′ 6″	....	....	244.00	Cost of passes.
Belgian Meuse....................	8′ 3″	$75.00	$78.00	153.00	Cost of weirs.
La Mulatière, Lyons.............	11′ 10″ (max.)	508.50	91.50	600.00	Cost of pass.
Kanawha River, W. Va., Dam No. 7, 1893.......................	8′ 0″ (13′ on pass sill)	226.00	26.80	252.80	Average of pass and weir.

COST OF CHANOINE WICKET DAMS WITHOUT SERVICE BRIDGES.

Upper Seine dams, prior to 1865....	5′ 11″	....	....	$88.50 $183.00	Cost of weirs. Cost of passes.

CHAPTER VIII.

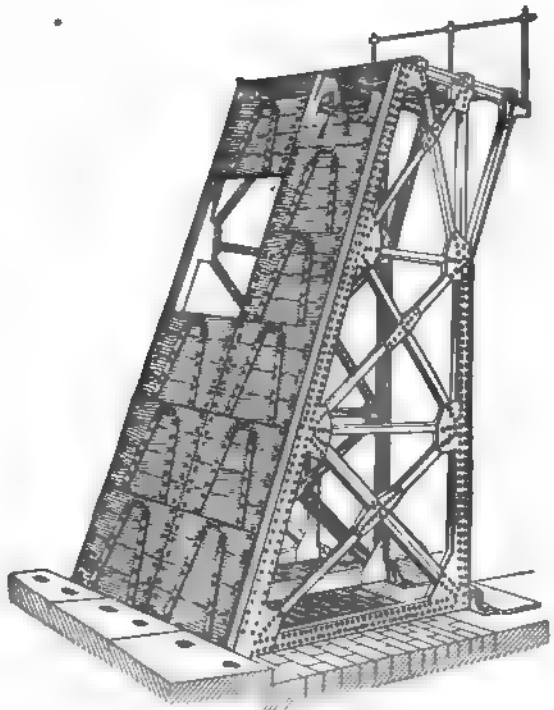
GATE, CURTAIN, AND BRIDGE DAMS. SHUTTER AND A-FRAME DAMS.

GATE DAMS.

General.—The Boulé gate, as it is called, was first used by M. Boulé, the French engineer, at the Port-à-l'Anglais dam near Paris in 1875, and has since been applied to important works in France, Russia, and elsewhere. It is a modification of the needle dam, the vertical needles being replaced by horizontal planks, the ends of which rest against and slide upon the faces of the trestles. The latter are hinged to the floor and are raised and lowered in a manner similar to trestles for needles.

Maneuvers. — To raise the dam, the trestles are first set upright, and connected by the floors and service track, and the gates are brought out on a truck and set in place by a light movable crane, or, where small enough, by a pole with a hook on its end. The regulation of the pool is effected by maneuvering the top row of gates, which are usually made from 4 to 12 inches in width, to permit easy movement. The lowering of the dam is done by reversing the operation of raising.

Dimensions, etc. — The trestles for this type of dam are usually spaced from 3¼ feet (one meter) to 4 feet apart, with a service bridge 9 to 18 inches above the pool, and are considerably heavier than those for needle dams. Their up-stream face is made smooth, and occasionally provided with a projecting rib, serving as a guide for the gates in sliding. The latter vary in width from 3 to 4 feet for the bottom ones, to between 4 inches and 21 inches for the top ones, the maximum area rarely exceeding 20 square feet. They are made of wood, strapped with iron, and are provided with a handle for maneuvering. An example of iron gates is to be found at the Pretzien dam on the Elbe (1875), and also at the new dam at Mirowitz, in Bohemia, where buckled-plates are used, sliding on rollers to reduce the friction. This appears to be an excellent device, as experiments made at the Marolles dam in France, with gates provided with ball-rollers, showed that one man



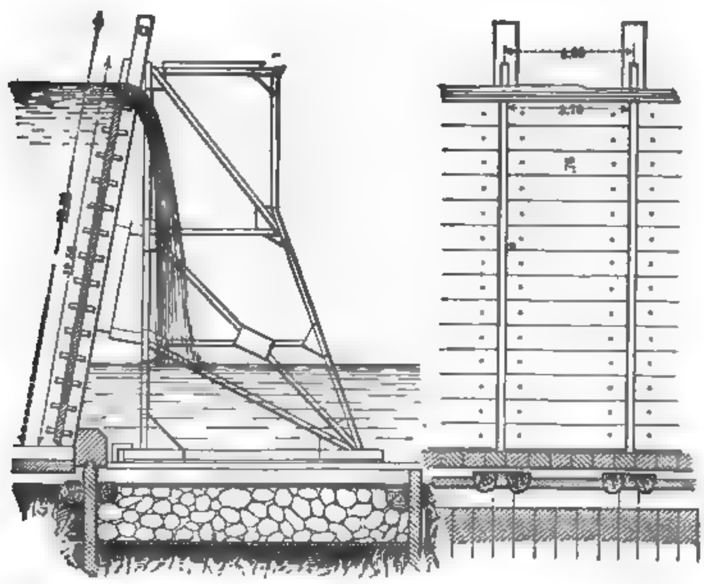
PERSPECTIVE VIEW OF BOULÉ GATES AND TRESTLES

could easily maneuver a gate of 5.8 square feet under a head of 7½ feet.* Ordinary rollers were also experimented with, but gave less satisfactory results. The effort required to lift the ordinary sliding gate under pressure reaches in certain instances as much as 2000 pounds.

Large sluice-gates moving on rollers have been used in England since 1876, where they were introduced by Mr. Stoney, an English engineer, and are known as the Stoney gate. They have been used up to 16 feet in width, some of the most recent examples being found on the great reservoir dams of the Upper Nile.

On the Libschitz dam (Bohemia, 1900) with 14 feet 9 inches on the sill, the Boulé gates used were five in number for each bay, four being 3½ feet high, and the top one 1 foot 9 inches high. The woodwork of the lowest one is 5 inches thick. The trestles are spaced 1½ meters apart, and are 19 feet 8 inches high, and weigh about 3000 lbs. each. The recess behind the sill is 3½ feet deep.

On the Boulé dam, at the head of the Louisville canal, Kentucky, which is at present the only dam of its class in this country, the trestles are spaced 4 feet apart, and are 7 feet high, and are composed of 1″×4″ bar iron. The depth on the sill is 5 feet, and the lowest gate is 2 inches thick. The dam consists of three openings, each 200 feet long, and one opening about 50 feet long. It was built in 1899, and is used as a flushing weir for the basin at the head of the canal.



Dam composed of Boulé Gates and Needles, as Used on the Moskva, Russia.

On the Moskva, in Russia, a combination of needles and gates is to be found, constructed about 1876. The needles are placed immediately above the trestles and are supported as usual, and the gates, which are planks 9½ inches wide and of a length equal to the width between each trestle, are laid horizontally, being supported by the needles instead of by the trestles.

Remarks.—The Boulé gate has proved to be an excellent device, as it is simple, and with proper appliances can be maneuvered under the full head of water. It is also cheap in construction, as it requires no great width of foundation, having in this respect an advantage over the Chanoine dam, which is almost always provided with the service bridge for operating. Comparative estimates of the two types for the

* Annales des Ponts et Chaussées, April, 1896.

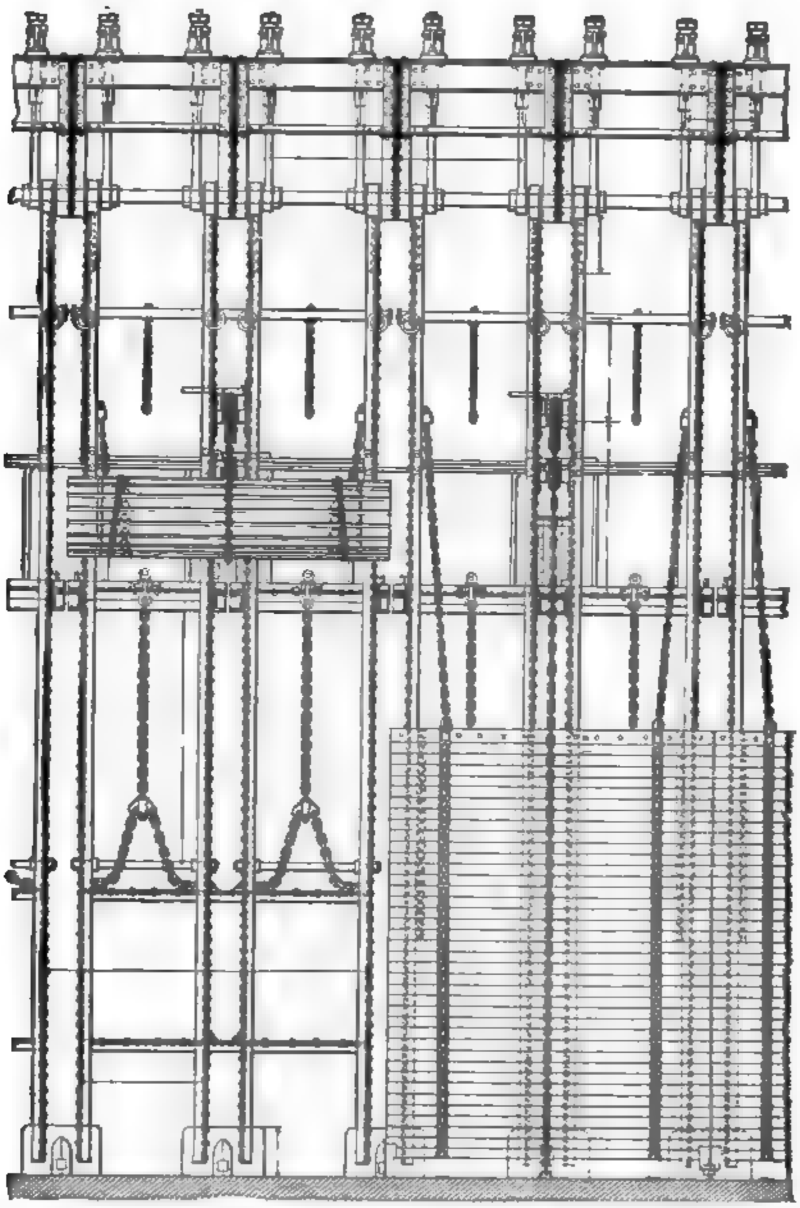
General View of the Libschitz Dam, Bohemia, in Operation, Showing the Method of Removing or Placing the Boulé Gates by a Traveling Hand-crane

The view is taken from the lock, and shows the pass in the foreground, with the weir and the raft chute on the other side. The pass is 213 feet long, with 14.7 feet on the sill, and is closed with Boulé gates. The weir is 160 feet long, with 10.4 feet on the sill, and is closed with needles. The lift is 12.7 feet. The work was completed in 1900.

(To face p. 243.)

dam at Port-à-l'Anglais, on the Seine, showed a difference of 30 per cent in favor of the former. These gates also minimize the dangers of scour, since the discharge is from the surface of the pool.

The principal objection to the Boulé dam is that it is comparatively tedious in



UP-STREAM ELEVATION OF A CURTAIN DAM, AS USED WITH AN OVERHEAD BRIDGE.

operation, each gate having to be handled separately and perforce slowly. To remove a gate from the top rank takes one or two minutes, and to remove one from the bottom rank, five or six minutes. For this reason the type is not adapted to rivers subject to quick rises.

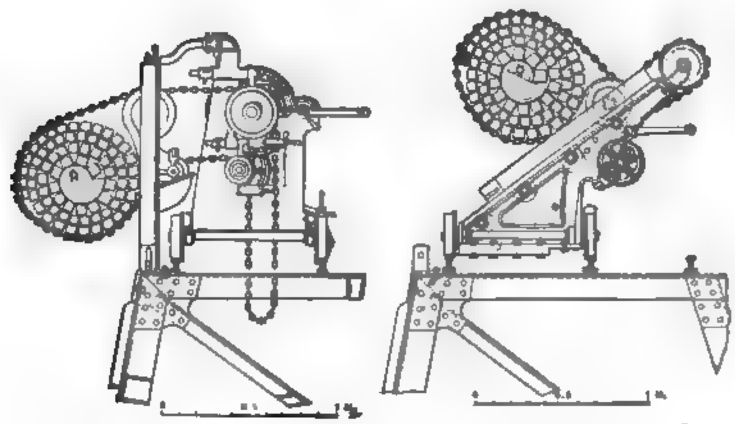
CURTAIN DAMS.

General.—The Caméré curtain is the invention of the French engineer M. Caméré. It consists of narrow horizontal strips of wood, hinged together, and capable of being rolled up by a chain, which passes round them, each curtain reaching from the surface of the water to the sill. (See illustration on page 243.)

This type is in use at Suresnes, Villez, Poses, and other important dams in France, where it has given excellent satisfaction.

Maneuvers.—The curtains are supported by trestles, of design similar to those for Boulé gates, and similarly raised and lowered, or by frames suspended from an overhead bridge and resting against shoes. When the dam is to be closed the curtains, each of which is attached to a frame, are brought out on a truck, and set in place between the trestles by a special winch and unrolled by releasing the chain. The pool is regulated by rolling them up from the bottom as much as may be desired. When the dam is to be opened they are rolled up entirely, and the frames with the curtains attached are lifted off and placed on the truck, and taken to the storehouse.

Remarks.—The Caméré curtain possesses an advantage over the Boulé gate in that it does away with the sliding friction of the latter, and hence can be used for higher lifts, that of the Poses dam being 13.7 feet, with 16.4 feet of water on the sill of the deepest pass. On the other hand, the curtain is more expensive than the gate, and more liable to get out of order, owing to its more complicated parts. It permits, however, of a very exact and easy regulation of the pool.



METHOD OF HANDLING CURTAINS WITH A SERVICE TRUCK AND CRAB.

The Suresnes Dam.—The Suresnes dam, located just below Paris, was built by M. Boulé in 1885. It is one of the largest as well as one of the most recent works on the Seine, and one of the best examples of movable dams in the world. It is composed of alternate bays of Boulé gates and Caméré curtains. The trestles of the pass are 19½ feet high, and the gates 17 feet in total height and 4.1 feet wide, supporting a head of 10.6 feet. The dam is divided into three parts by two islands. The navigable pass is 238 feet wide, and is located in the left arm of the river, while in the right arm there is an elevated pass 206 feet wide. Between the two islands there is a weir 206 feet wide. The sills of these three passes are 17.9, 16.25, and 12.13 feet, respectively, below the level of the upper pool.

The gates and curtains are supported by Poirée trestles 19.5, 17.9, and 13.44 feet

View Showing the Lowering of the Pass Trestles, Libschitz Dam, Bohemia.

(To face p. 244.)

high, weighing respectively 4000, 3000, and 1760 lbs. They are constructed of channel iron, with two uprights front and back, joined by cross-bars and braced by a St. Andrew's cross. A continuous chain is used to raise and lower the trestles, operated by a windlass on the shore, and they can be maneuvered six at a time.

In the right arm of the river, *i. e.*, in the elevated pass, Caméré curtains are used for closing the dam. The middle channel or weir is closed with Boulé gates, and the left arm or navigable pass is closed with gates and curtains alternately. The last method of arrangement has proved the best, for the reason that the curtains when rolled up do not always preserve a cylindrical form. This fact is likely to cause adajcent curtains to jam.

In time of flood the top rows of gates are first removed, and then the curtains are rolled up. If the flood continues, the gates are all removed and the trestles in the navigable pass are lowered. Thus the whole pass is opened to navigation with a depth of 10½ feet on the sill. The time required to either raise or lower the trestles of the pass (fifty-seven in number) is three hours. As it involves a considerable delay to navigation to pass boats through the lock, the trestles are always bedded as soon as there is sufficient water on the sill for the purpose of navigation. If the river falls, the trestles are immediately replaced in order to preserve a proper stage of water. In the other passes, which are but little used for navigation, the trestles are not lowered until almost submerged. The curtains are removed by means of a special car carrying a small windlass, and are carried to a platform on shore and hung upon a special frame in the same position as the dam. The operation is laborious, as each curtain and frame weighs 1600 lbs. As they are rolled up, débris of various kinds catches between their sticks, and they have to be unrolled and cleaned.

The method of removing gates is to pile them on a truck rolling on the bridge, when they are taken to the bank and stacked there.

A new lock, 525 feet long and 56 feet wide, was placed between the left bank and the old lock. The latter was rebuilt, and below it a lift-wall was placed. This was continued by a new lock 165 feet long.

The cost of the dam is given as $632 per lineal foot, all included.

Calculations for Trestles for Gates or Curtains.—Trestles for use with the Boulé or Caméré types of dams have to support in addition to the direct strains, the bending from the water-loads on the up-stream leg.

To determine the dimensions required, first find by moments or by graphics the strains in the various pieces AB, AG, etc. (Fig. 21) induced by the water-loads P_1, P_2, etc., considered as concentrated at the panel points A, B, etc. This will give the maximum strains in the web and down-stream members, but the up-stream member $ABCD$ has to support in addition, between each panel point, the bending from the gates or curtains, which rest directly on it. In all ordinary cases economical manufacture will require $ABCD$ to have the same section from top to bottom, as, for instance, two chan-

nels and a cover-plate, so that only the maximum bending moment, which occurs in *CD*, need be found. One-fourth may be deducted from this moment, as *AD* acts as a continuous beam, and the result used in the formula below.

The total area of section required will then be

$$A = \frac{1}{S}\left\{d + \frac{Mc}{r^2}\right\},$$

where A = total area required; S = total allowed strain per square inch; d = amount

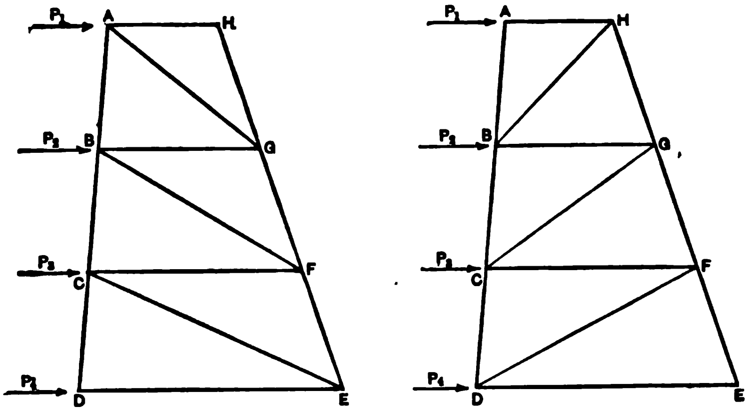


FIG. 21.

of direct compression or tension in the member; M = maximum bending moment in inch-pounds; c = distance in inches of extreme fiber from neutral axis; r = radius of gyration of the section.

The gates or curtains themselves have to sustain the pressure of the water, which equals for any gate

$$w \times h \times d \times 62\tfrac{1}{2} \text{ lbs.} = P,$$

where w = horizontal length of gate, h = its height, and d = distance from surface of water to center of gate, all in feet.

The bending moment is $\dfrac{P \times w}{8} \times 12$ inch-pounds $= M$, and $M = \dfrac{SI}{C}$, where S = extreme fiber stress per square inch, C = half the thickness of the gate, and I = its moment of inertia.

DIMENSIONS OF TRESTLES FOR GATE AND CURTAIN DAMS.

Location.	Height.	Width of Base.	Ratio of Base to Height.	Distance, C. to C.	Lift.	Weight, Each, lbs.	Remarks.
Villez, Lower Seine........	17′ 9″	14′ 9″	$\frac{83}{100}$	....	9′ 10″	4360	Curtain dam.
Suresnes, Lower Seine (1885)	19′ 9″	12′ 4″	$\frac{62}{100}$	4′ 1½″	10′ 8″ (15′ 0″ on sill)	4000	Gates and curtains.
Libschitz, Bohemia, Moldau River (1900)............	18′ 11″	12′ 6″	$\frac{66}{100}$	4′ 1½″	12′ 10″ (14′ 8″ on sill)	3750	Gate dam. Has two floors, side by side, each 30″ wide and 1′ 8″ above pool.

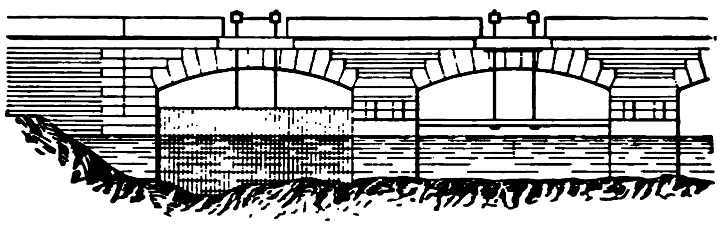
VIEW SHOWING TRESTLES FOR BOULÉ GATES AT THE LIBSCHITZ DAM, BOHEMIA, TAKEN DURING CONSTRUCTION.

The trestles are 4 feet 1 inch apart and 19.7 feet high, weighing, with floors and all attachments, 3740 lbs. each. The clamp for the operating chain is seen on the top member (see p. 270).

(To face p. 246.)

BRIDGE DAMS.

The bridge dam invented by M. Tavernier and employed at Poses, Mericourt, Meulan, and Port Mort, in France, while more expensive than those heretofore described, yet may be advantageously used in many situations, particularly at points where a railway or highway is to cross a stream. While the application of dams supported by bridges is of late date, yet M. Frimot proposed such an arrangement in 1829, and a bridge dam was constructed on the Upper Yonne as far back as 1836. These designs, however, did not provide for navigation under the arches, except when the dam was open and the water low.



BRIDGE DAM ON THE YONNE, FRANCE. 1836.

On the river Elbe, at Pretzien, in Prussia, a bridge dam was built in 1874–75. It differs from those in France in that the closing is done with sliding gates instead of hinged curtains. It is built with nine bays, 41 feet wide, separated by piers on which rests the bridge, which is too low to permit boats to pass underneath, even when the dam is open. The floor of the dam is above low-water level. The pool level is 10 feet above the sill.

Poses Dam.—The following description of the dam at Poses on the Lower Seine will give a general idea of the system. This dam, which was completed in 1885, and has a lift of 13.7 feet, measures 771.5 feet between the abutments, and is divided into seven passes, five deep and two shallow ones.

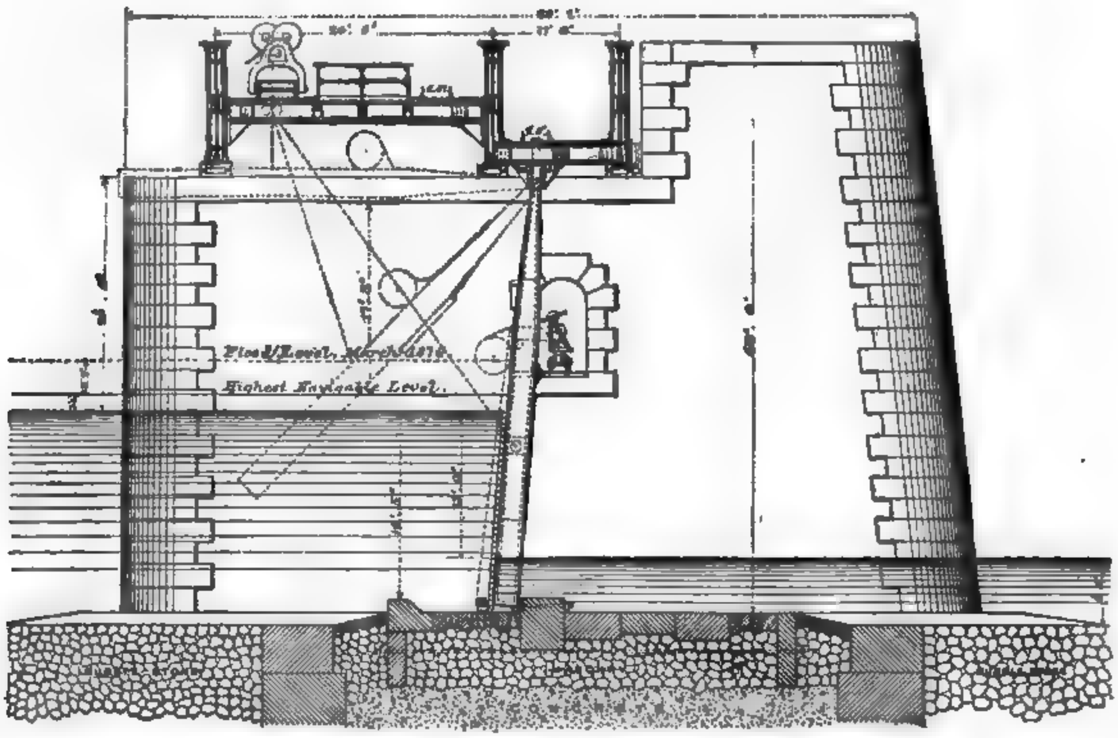
The system adopted required the establishment of two upper bridges, first, the down-stream bridge to hold the suspended frames, and second, the up-stream bridge to hold the windlasses while the frames are being raised, and also to sustain a part of the weight of the raised frames themselves. The first may be called the suspending, and the second the hoisting, bridge. Two of the passes are arranged so the spans can be moved in time of floods, to permit the passage of boats.

The roadways are for two different purposes, and at different levels. The down-stream longitudinal girder of the hoisting-bridge is omitted, and its supporting cross girders are attached directly to the longitudinal up-stream girder of the suspending-bridge, thus affording easy communication between the bridges, and adding to the horizontal strength of both. The up-stream roadway has an opening 4.99 by 8.20 feet; large enough to admit of passing the curtain through it endwise. In the non-navigable passes facility of communication is insured by putting a third roadway above the beams of the down-stream roadway.

The lattice girders supporting the roadway have their uprights 7.61 feet apart, corresponding to the widths of the moving parts. The cross girders take the strain of the hanging-frames by means of the brackets arranged under them. These girders, 3.80

feet apart, are braced by channel-irons placed on each side of the rods suspending the frames. The brackets are trapezoidal in form and 2 feet high. Upon each of their faces two angle-irons are riveted, projecting on each side and forming a guide. The heads at the ends of the suspending bars rest upon these guides, 1.64 feet under the cross girders, so that the uprights can be raised to the flanges.

The width of the up-stream roadway depends on its height above the water. There must be space enough, from the end girder to the point where the chain comes through, to work the windlass; also to give the chain a proper inclination, to avoid too much tension on it. At Poses the chain is attached to the frame at 2.95 feet below the water-



General Section of Poses Dam.

level, and the chain is inclined 33° at the beginning. The distance between the principal girders of the up-stream roadway is 24.76 feet for the navigable passes, and 17.22 feet for the non-navigable passes.

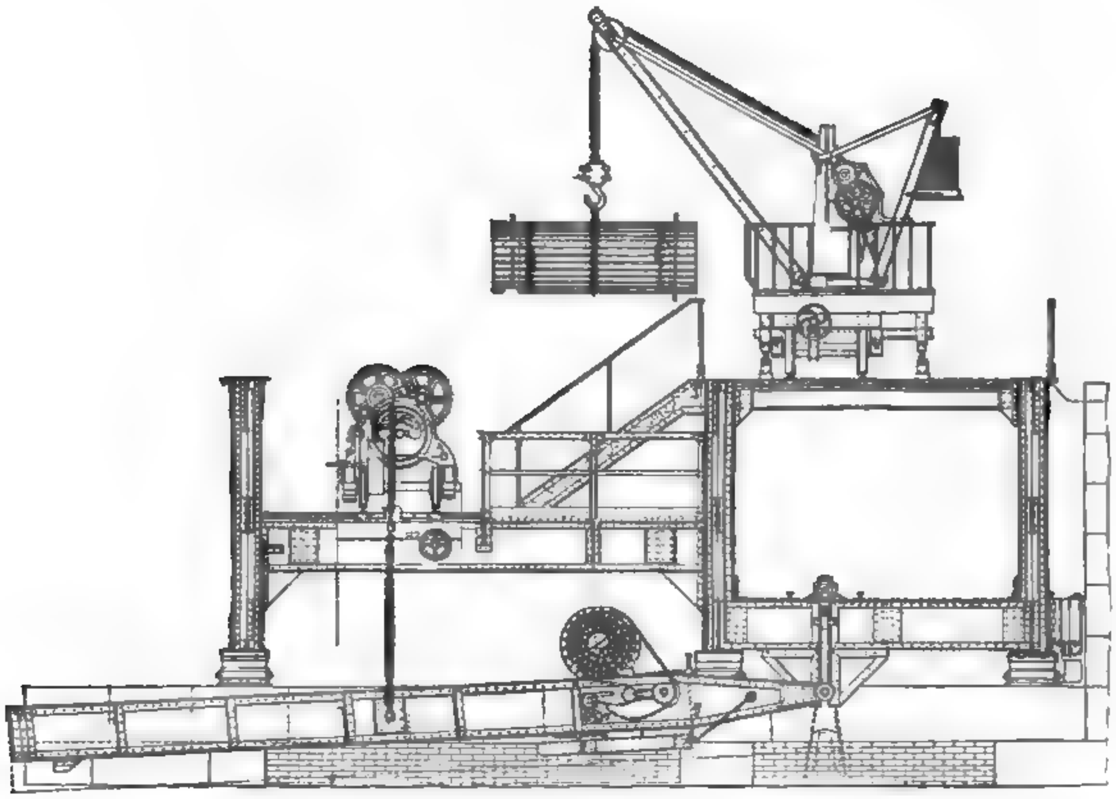
The up-stream roadway is placed half-way up the principal girder, so as to allow sufficient space below the cross girders to store the rolled curtain when the frames are raised.

The frames, which support the curtains, are wrought-iron girders, inclined so that the vertical passing through the center of gravity of the frame with its curtain and foot-bridge is on the up-stream side of its upper joint. They have an I-shaped section, which is constant in width for 8.20 feet above the upper pool for the same pass; this width is 1.64, 1.96, and 2.30 feet for the three passes, respectively. Above this level the width tapers to 0.82 foot at the top.

The joint with the suspender bars is made by a cast-steel eye riveted to the web

of the frame. Lengthwise the frames are arranged in groups of two, braced together. and the axes of these groups are 3.80 feet apart. The object of this division was to reduce the width of the curtains to 3.80 feet, in case the width used, 7.61 feet, should be found too great; but as it has been found convenient, the arrangement of the frames in subsequent dams of this type has been simplified.

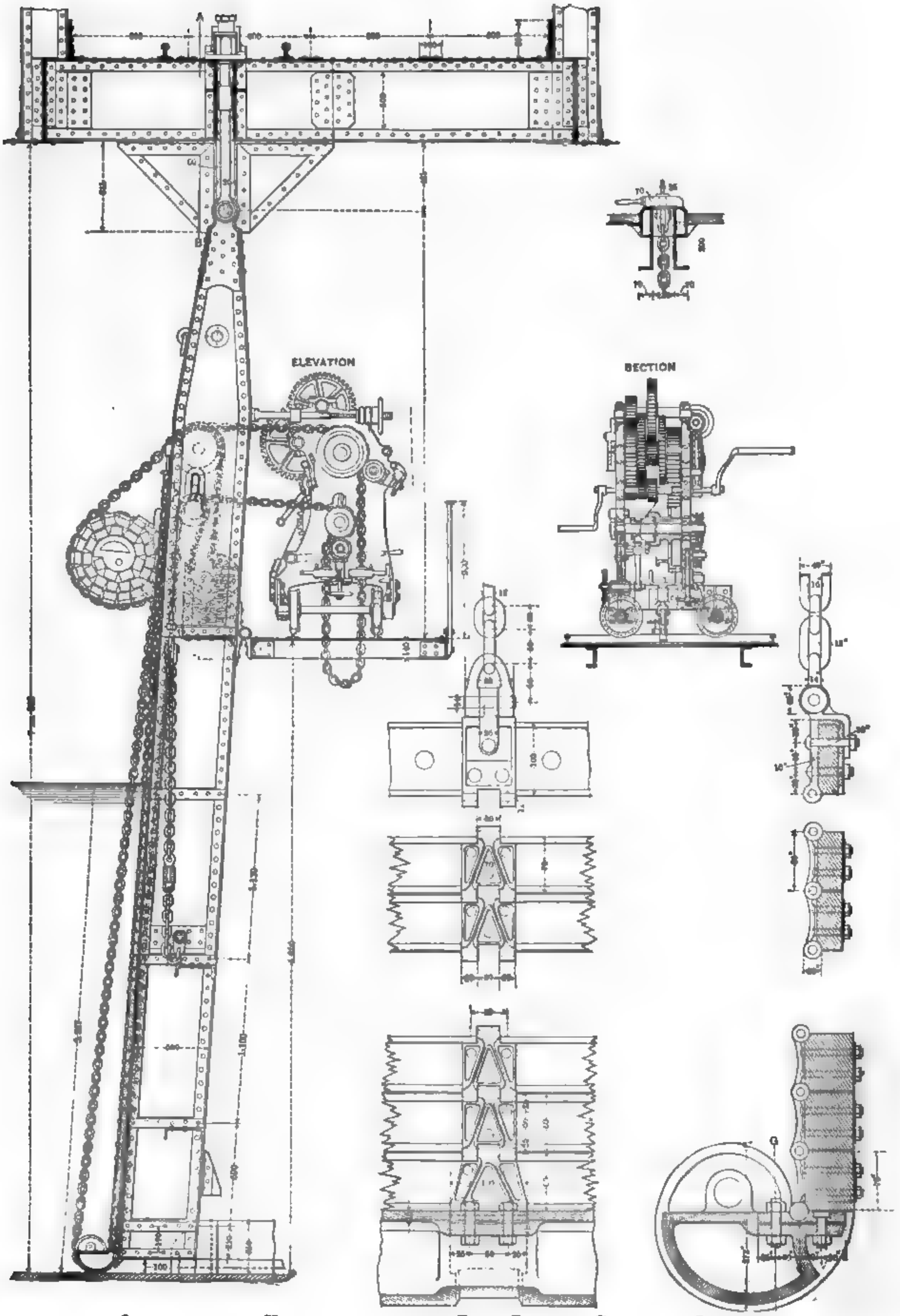
There are two hoisting-chains for each frame, and each chain is divided into two branches, the end of each branch being attached to an upright, thus dividing the strain lifting the frame into four equal portions. On the down-stream side of the uprights a strong wrought-iron hook with angle-irons is attached, for the purpose of raising the frame in case of accident to the chains or to their attachments. This can be done by lowering, along the upright, a chain, the bight of which will be held



Section showing Bridges and Operating-crane at Poses Dam.

securely by the hook. Ring-bolts are attached to the up-stream side of the uprights, so that the frames may be slung below the upper bridge when any repairs are required.

The suspending chains for the curtains are hooked to rings attached to the two outside uprights of each frame 4.10 feet above the foot-bridge. The two pulleys for rolling the curtains are placed between the intermediate uprights. The lower pulley, holding the down-stream chains, is slightly smaller than the other. This inequality insures a distance between the chains equal to the thickness of the first curtain bar. Besides rolling the curtain, each side of the endless chain can be fixed upon its guide-pulley by a stop carrying a finger, which enters the link of a chain when



SECTION SHOWING UPRIGHTS, ETC., OF THE POSES DAM, AND DETAILS OF CURTAINS.

the lever is lowered. Finally, the uprights have on their up-stream faces iron claws, which serve as stops to the rolled curtains.

Each curtain corresponds to an opening 7.61 feet wide and 17.55 feet high in the deep passes. The bars are of yellow pine, each 0.25 foot high, with one-sixteenth of an inch play between them to allow for swelling. Their length is 7.47 feet; giving a play of 0.13 foot between two neighboring curtains. This interval can be closed by a joint cover if the dam requires to be made tight. The thickness of the upper bar is 0.13 foot, and it increases progressively downward. The upper bar, exposed to shock from floating bodies, is strengthened by an angle-iron. The hollow cast-iron rolling shoes are heavy enough to cause the curtain to sink easily into the water when unrolled. The rows of hinges are of bronze, so as not to rust. They have strong flanges, and their pins are of drawn phosphor-bronze. All the handling machinery can be carried on cars rolling on the service-bridge tracks.

With the suspension above described and in use at Port Mort the frames can be removed for repairs as follows: Lifting-jacks are placed under the cross-pieces uniting the two suspending rods of a frame above the down-stream roadway. Each jack rests upon a platform arranged for this purpose in the horizontal bracing of the roadway. After placing the jack and removing the wedges which prevent the lifting, the jack is screwed up, care being taken to wedge the ends of the cross-piece as it moves up. This wedging serves to sustain the lifted frames. The chains from the windlass on the upper bridge are then hooked on, and the frames are rotated to a horizontal position and made fast to the under side of the upper bridge.

The dam is operated by four electric cranes, traveling on the bridges, the power being supplied by turbines.

The cost per foot run was as follows:

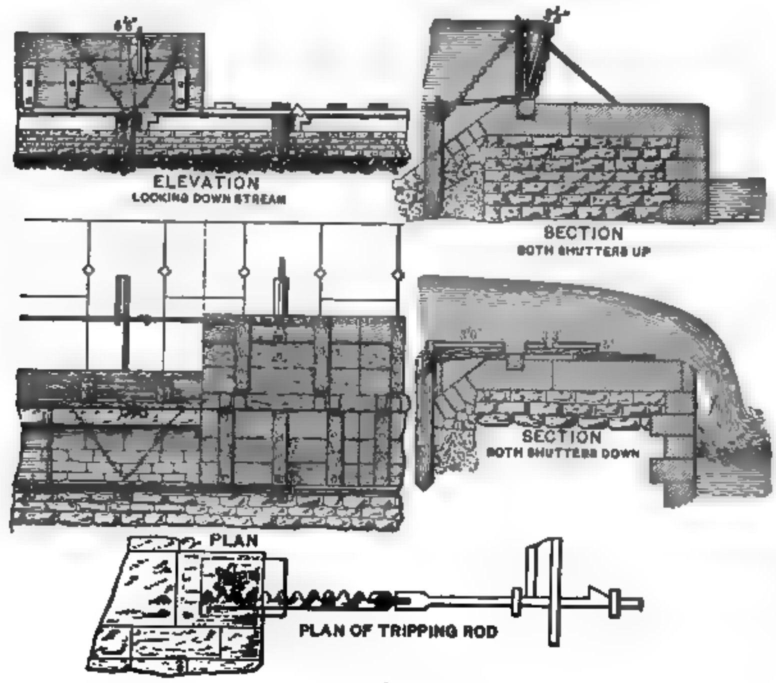
Masonry foundations	$813.72
Ironwork:	
Upper bridges	114.09
Frames	53.54
Curtains, etc.	25.68
Total	$1007.03

The employment of the existing type of bridge dam on navigable streams in America is not a probability, because navigation requires considerable "headroom," and most of the rivers vary too greatly between low and high water. On the Ohio, for example, few bridges are less than 100 feet above low water.

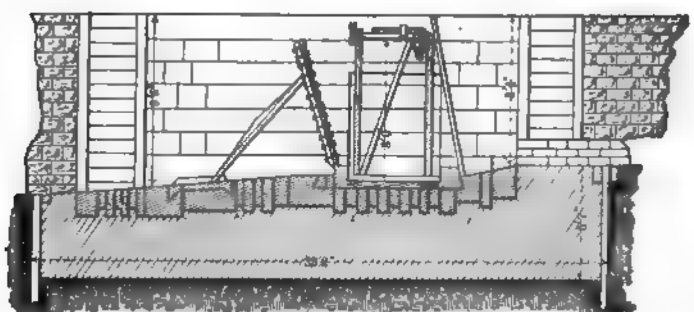
SHUTTER DAMS.

General.—The development of the shutter dam is due to M. Thénard, who conceived the idea of the type from certain weirs on the river Orb, in France, which had been provided with small movable shutters during the eighteenth century.

The Thénard shutter consists of a panel hinged to a floor at the bottom and supported by a prop near its middle on the lower side, the shutter lying down behind a sill when lowered. To facilitate the lowering M. Thénard introduced a bar provided with projections, and capable of being pulled or pushed by means of gearing in the masonry. It was arranged so that the projections would strike the props one at a time and pull them from their supports, thus allowing the shutters to fall. This was known as the tripping-bar. It was found difficult to raise them, however, against



DETAILS—THÉNARD DAMS.



SECTION-POIREE THENARD DAM

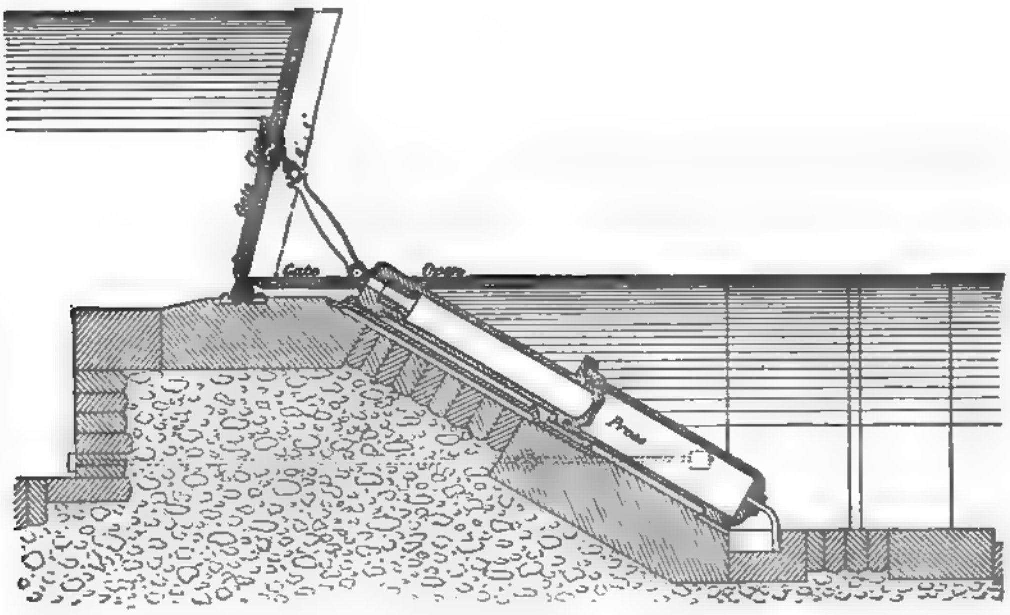
any head, and for that reason counter-shutters were introduced up stream. These were arranged to rise down stream, or in the opposite direction to the main shutters, being held in an upright position by chains.

Maneuvers. — To raise the dam, the counter-shutters, which were kept in place on the floor by latches worked by a tripping-rod, were released, and the force of the

current caught them and swung them over till they were stopped by their chains in a position nearly vertical. This dammed up the water, and the workmen then raised the main shutters by hand. The space between the two rows was then filled with water by opening curtain-valves, and the counter-shutters were pushed down and latched. The lowering was done with the tripping-bar.

Examples.— Four dams were built in France of this type, the last one, that of St. Antoine, having been completed in 1843. The shutters in this dam were 5.6 feet high and 3.9 feet wide. In the others they were 3.3 feet high and 6.6 feet wide, the length of the weirs being 156 feet and that of the passes 74 feet.

A dam with a pass closed by Thénard shutters was also built in 1850 at Courbeton on the Seine, but with needles and trestles substituted for the counter-shutters. The weir was closed by needles alone. The pass was 39.7 feet long and the shutters were lowered by a tripping-bar, worked by a turbine. The action of the latter was automatic, starting or stopping as the pool level rose or fell.



SECTION OF A GIRARD SHUTTER DAM.

Other examples are to be found in India, in the Mahanuddee and Cossye rivers, and in the Sone Canal, where they are used to close flushing-sluices. Some of them are over 21 feet in length and support heads of nearly 10 feet. Much trouble was experienced there with the breaking of the chains of the counter-shutters, due to the sudden jerk when the latter came to place. This was successfully overcome by using hydraulic brakes on the up-stream side, consisting of a piston working in a cylinder pierced with small holes, the piston being attached to the shutter, and the cylinder to the floor. As the shutter rises it draws the piston through the cylinder, and the water escapes slowly through the holes, thus checking the rapidity of its movement.

Girard Shutter.—To overcome the difficulty of raising the Thénard shutter M. Girard proposed to do away with the counter-shutters and to substitute for the props hydraulic jacks, which would raise or lower the main shutters at will. Seven of this type were constructed in 1870 at Auxerre on the Yonne, and gave excellent satisfaction. The shutters were 11½ feet wide and 6½ feet high, the cylinder of the jack being 12 inches in diameter. On the introduction or withdrawal of the power the piston-rod moved out or in, thus raising or lowering the shutter.

The cost of the dam per lineal foot was $60 for the fixed parts and $119 for the movable parts, or a total of $179.

Owing to the untimely death of its inventor no further examples of this type were constructed.

Remarks.—The shutter dam possesses the advantages of simplicity, and of not being easily injured by drift or ice, since there are few parts in which these can be caught. The difficulty of raising it, however, and the expense of the counter-shutters, proved a serious drawback, and after the introduction of the Chanoine wicket the type became practically obsolete.

A-FRAME DAMS.

This design relies wholly upon trestles for the retention of the water, as well as for the support of its pressure. The trestles may be raised and lowered precisely as are other trestles, or with a special arrangement similar to that used on the needle dam on the Big Sandy River at Louisa, Ky.

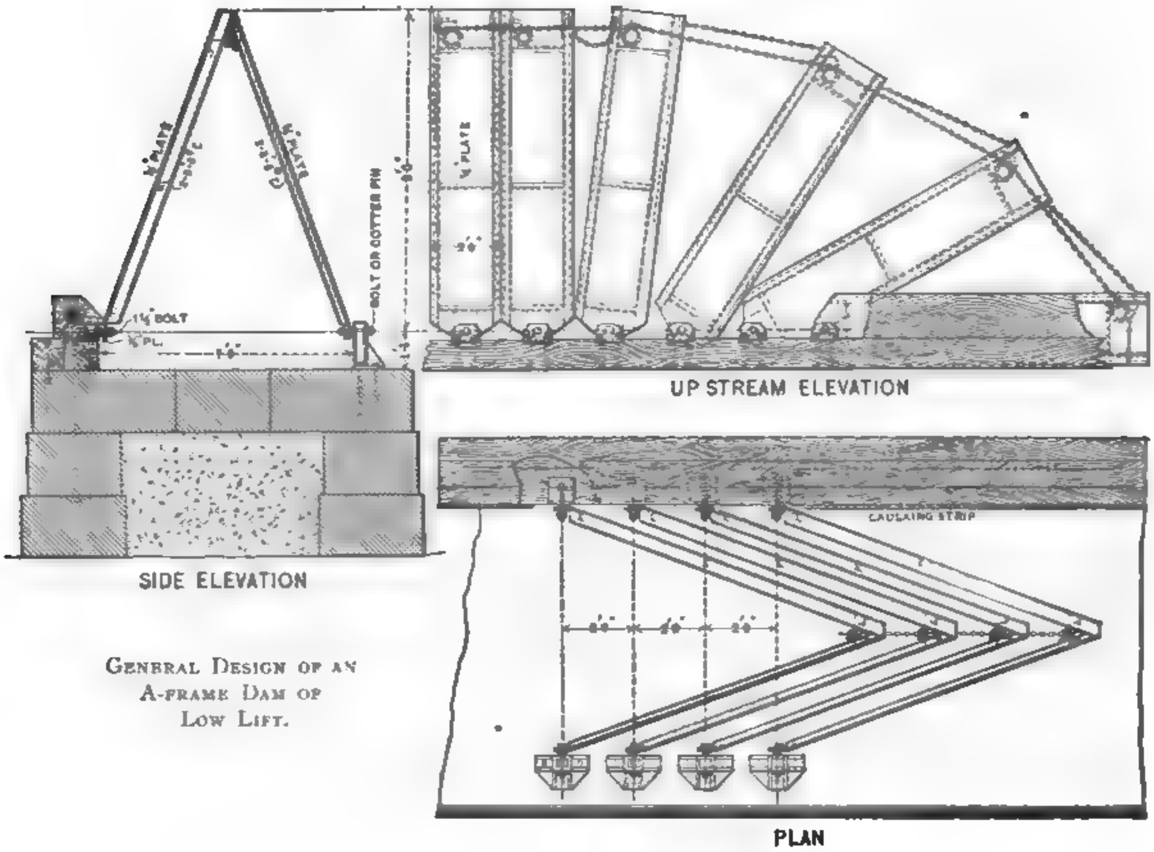
Description.—The dam consists of a number of A-shaped trestles set up adjoining each other across a stream, the up-stream faces or legs of which, in connection with a sill (which also protects the trestles when down), hold back the water. The two legs of each trestle are connected at the top, by plates forming a walk, and at the bottom they terminate in eyes which are connected by pins to journal-boxes and sills attached to the masonry.

The up-stream member of each trestle is a frame of channels suitably arranged and covered with plates riveted on. The edges of these plates touch each other on adjacent trestles when standing, and may extend slightly over the channels, or the latter may be set flush with flanges toward each other. The latter construction gives a thicker wall through which the water must pass in leaking. Wood may be inserted along the channels when exceptional tightness is desired. The down-stream post may be similar to the up-stream one, or it may be latticed, or even consist of a single member like the prop of a wicket. At the bottom of each post a piece with an eye, bent so as to have the part containing the eye stand vertical, is riveted to the frame, and these are connected by pins to journal-boxes on the floor. These boxes have their eyes centered at a greater distance from the floor than one-half the width of the trestle face, so that the trestles may turn without binding. The upper box is imbedded in the

sill, or may be a part of it. A space is left open between the lower boxes for the escape of water, and to assist in keeping the floor clean. The sill closely fits the upper face of the trestles at the point where the angle is made with the eye-pieces, so as to prevent leakage. The construction may be such as to permit overflow.

As the top of the trestle will be from 18 to 30 inches wide, it will answer for a walk as required.

In the head of each trestle is located a pocket-wheel for use in connection with the maneuvering chain. This wheel turns on a shaft attached to the frames. At one edge of the wheel, and forming a part of it, is a ratchet. A pawl, having a tooth which may fit loosely into this ratchet at one end, and having the opposite end formed into a rounded wedge, is pivoted so as to be readily lifted out of the ratchet by depressing



General Design of an A-frame Dam of Low Lift.

the wedge end. This depression takes place just as the trestles become vertical in raising, by the rounded end being pushed by a stop or projection on the adjacent trestle made for the purpose. As long as the trestles remain touching each other this stop will hold the tooth of the pawl out of the ratchet, and the wheel is free to turn. Let a trestle begin to incline or descend, and the pawl, being released from the stop, immediately falls into the ratchet and arrests the movement of the wheel. The pockets in the wheel are made to fit the chain used for raising and lowering the trestles, and this chain cannot move without also moving the trestle when the pawl is in the ratchet.

In place of the chain-wheel and pawl a latch can be used, placed and removed by hand at the time of the maneuvers.

In addition to the maneuvering chain, which may be connected with or disconnected from the trestles at will, there may be placed between each adjacent trestle a few feet of chain, called the fixed chains. These are fastened to the trestles by eye-bolts, and their length is determined by the number of trestles it is desired to raise simultaneously — that is, by the power of the crab· but they must be sufficiently long to permit the trestles to lie flat when down.

Maneuvers. — On the lock wall or pier is located a chain crab for maneuvering. The chains which pass over the pocket-wheels in the trestles are operated by this crab. The last trestles are made fast to the ends of the chains.

The methods of lowering and raising are the same for a pass as for a weir. To lower the dam the trestle next the abutment is unhooked from the masonry and pulled toward the abutment, the chain being unwound at the same time on the crab, until it tightens the fixed chain (in case one is used) between it and the next trestle and starts that trestle downward. As this occurs the pawl will engage with the ratchet, and lock the next to the last trestle on the chain. The unwinding goes on continuously, and when the next fixed chain is stretched it will start a third trestle, and so on until all are down. In case no fixed chain is used, a man standing on top makes the connection between the chain and trestle.

To raise the trestles the chain on the crab is wound in, bringing up the first one (being the last lowered) and starting several others. When the first becomes vertical and strikes the masonry, its pawl is lifted out of the ratchet by a stop made for the purpose, and thus the trestle is released from the chain without stopping the crab. The continuation of the winding brings up the second trestle, which is released from the chain when its pawl strikes the stop on the first. All trestles are thus raised, after the first, by winding in a length of chain equal to that of the short chains connecting the trestles, or equal to the spacing given when the dam was lowered. Where a continuous chain is not used the trestles may be raised and lowered precisely as are those of wicket and needle dams, each trestle being connected with its neighbor as brought up.

To regulate the pool, in either case, it is only necessary to lower a sufficient number of trestles on the weir next the abutment. In these, which are most liable to be used for pool regulation, the fixed chains may be lengthened so as to give less load on the crab in order that the operation of raising may be performed by the watchman alone when necessary.

Advantages Claimed.—The advantages claimed for this style of dam over those formed by needles, gates, or wickets are:

The dam, being raised and lowered across the current, can be operated either wholly or partially under great heads of water with two or three men.

In raising, the dam is complete when the trestles are up, while in other forms it

View of Bear-trap Weir, Dam No 6, Ohio River (1901).

The bear-trap is raised in position and seen from the down-stream side. The bottom sheathing planks are yet to be put on.

View of A-frame Weir, Dam No. 6, Ohio River (1901), Showing the Method of Raising or Lowering the Frames.

(From sill to pool is 13 feet 2 inches.)

(To face p. 257.)

has but commenced. It is therefore more rapidly raised; the lowering is also more rapid.

There is nothing left standing to catch drift after the lowering begins.

There are no extra parts to care for when lowering or when not in use.

There is no danger to operatives in the maneuvers.

As there is no double construction, the foundation is narrow; hence the cost is reduced.

The leakage will be very little, as there are few joints.

If submerged, it will act as a fixed dam without injury to itself.

A weir of this character, 120 feet long and 13 feet 2 inches high, is in use at Dam No. 6 on the Ohio River.

Calculations. — The strains in a trestle of the A-frame style are composed of compression in the down-stream leg and tension and bending combined in the up-stream leg. It may be mentioned that as the height of these trestles is increased, necessitating an increased proportion in the width of the base, the resultant of the water pressure will fall within the latter, thus removing any upward pull on the anchorages.

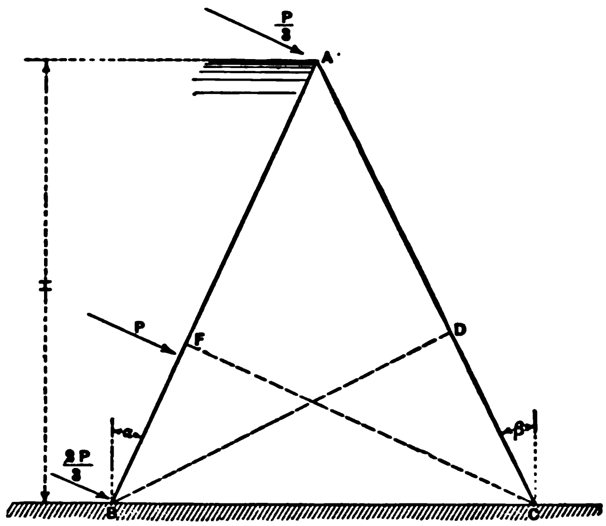


FIG. 22.

Let ABC (Fig. 22) represent an A-frame of width of face w, supporting a head of water H, and with legs inclined at angles α and β as shown.

The pressure P on the up-stream leg is $wH \sec\alpha \cdot \frac{H}{2} \times 62\frac{1}{2}$ lbs.

Taking moments about B, we have

$$P \times \frac{H \sec\alpha}{3} = AC \times BD, \quad \text{or} \quad AC = \frac{PH \sec\alpha}{3BD}.$$

Similarly, taking moments about C, if CE is the perpendicular from C to the direction of P,

$$P \times CE = AB \times CF, \quad \text{or} \quad AB = \frac{P \times CE}{CF}.$$

These equations will give the direct strains in AC and AB, but AB has to support in addition the bending from the water. The point of maximum bending for a beam supporting water level with its top and on one side only occurs at $\frac{H}{\sqrt{3}}$, H being as given above. The moment thus is

$$M=\frac{P}{3}\times\frac{H}{\sqrt{3}}\sec a-w\times\frac{H\sec a}{\sqrt{3}}\times\frac{H}{2\sqrt{3}}\times\frac{H\sec a}{3\sqrt{3}}\times 62\tfrac{1}{2}\text{ lbs.}$$

$$=\frac{wH^3\sec^2 a\times 62\tfrac{1}{2}\text{ lbs.}}{9\sqrt{3}}\times 12\text{ inch-pounds.}$$

The total section required is then found by the formula

$$A=\frac{1}{S}\left\{d+\frac{Mc}{r^2}\right\},$$

where A = total area required; S = total allowed strain per square inch; d = amount of direct compression or tension in the member; M = maximum bending moment in inch-pounds; c = distance in inches of extreme fiber from neutral axis; and r = radius gyration of the section.

The angles a and β can only be determined by trial, as there is usually only one position for each height of trestle which will give the greatest clearance when the dam is lying down.

AB and AC should be strongly connected at the top with deep gusset-plates, as, if the legs are wide apart there, secondary bending stresses will be introduced, causing a tendency to buckle.

CHAPTER IX.

DRUM WICKETS AND BEAR-TRAPS.

DRUM WICKETS.

THE drum wicket was invented by M. Desfontaines, and applied first in 1857 at Daméry, on the Marne, a tributary of the Seine. After its success was assured, others were built on the same river, the last one being that of Joinville, finished in 1867. Other examples are to be found in Germany, generally as sluices, their lengths varying from 17 feet to 39½ feet, with depths of water on the sills from 5.6 feet to 9.2 feet. Only one example is to be found in America, of modified design, that upon the Osage River, in Missouri, where a total opening of 750 feet is closed by 10 wickets, each 75 feet long, in one piece, the sections being separated by masonry piers. The depth on the sill is 7 feet. There has also been built a lock-gate of similar design, for lock No. 2 of the Mississippi River, near St. Paul.

Description.— The Desfontaines wicket consists of two diaphragms in approximately the same plane, one above and one below, joined together and turning on a horizontal axis. The upper one is exposed; the lower one works in a closed chamber which is filled or emptied by culverts closed with valves. To raise the dam the water from the upper pool, which must have more or less head, is admitted to the chamber, and by pressing on the lower diaphragm, causes the whole wicket to revolve, thus raising the upper diaphragm. To lower the dam, the water in the chamber is allowed to escape, and the water of the upper pool pushes over the upper diaphragm and forces it down behind its sill. The operating valves are always placed on shore, so that the pool can be regulated from the bank. The lower arm is usually made of the same length as the upper one, or a little longer.

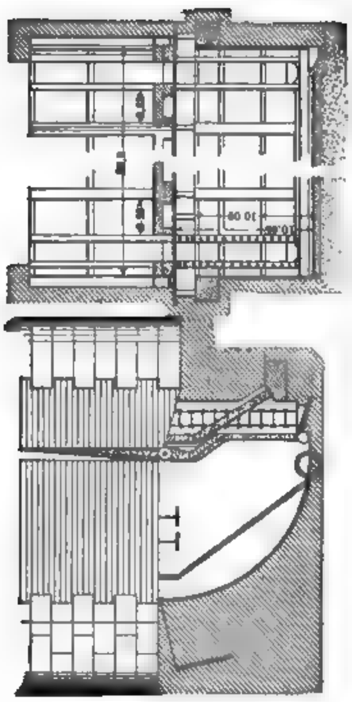
Owing to the existence of the chamber, which requires a certain depth of foundation, and to the necessity of a head of water for operation, these wickets have only been used for weirs.

Joinville Dam.—The pass of this dam is 39.4 feet long, and is closed by needles. The weir, which has a length of 206.7 feet, is closed by 42 drum wickets 3.3 feet high above the sill, placed side by side, each wicket being a little less than 5 feet long. They are separated in the recess by cast-iron diaphragm plates, built in the masonry and pierced for the openings required for the admission and exit of the operating water.

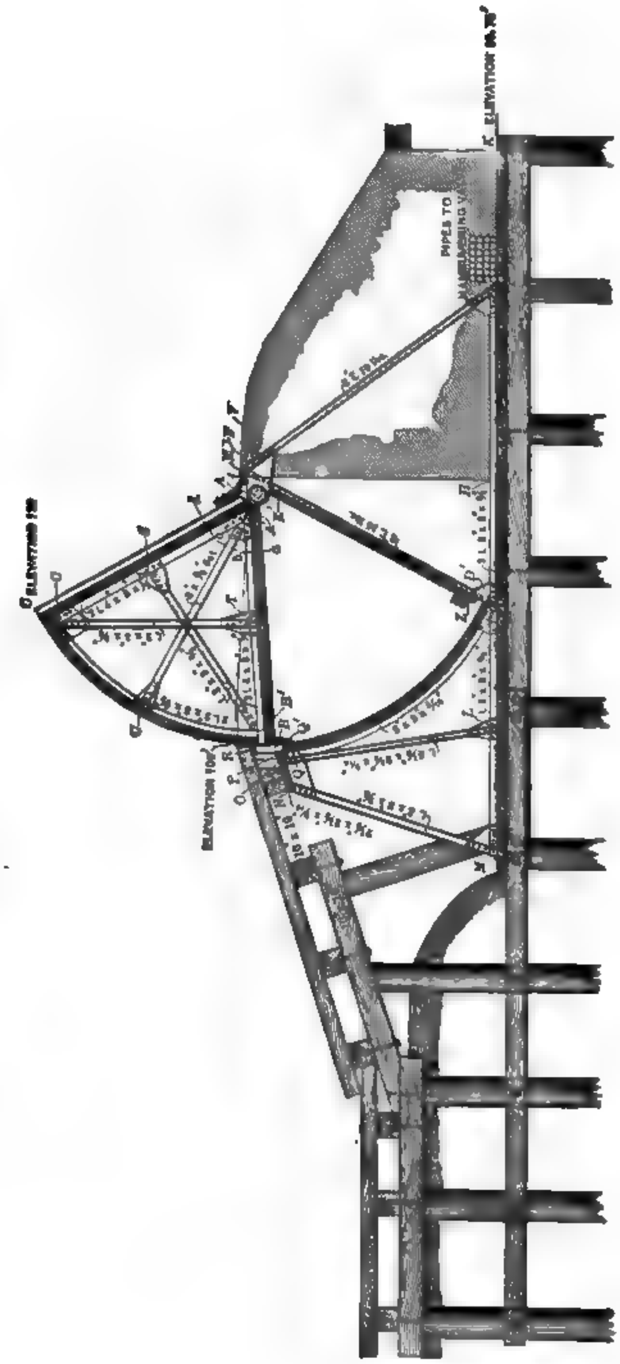
The entire weir can be raised or lowered in a few minutes, and against a full head of water, one wicket rising or falling after another until all are maneuvered.

The cost of this weir per lineal foot was $105 for the fixed parts and $48 for the movable parts, or a total of $153.

Remarks.—The Desfontaines wicket is the simplest and most easily operated of



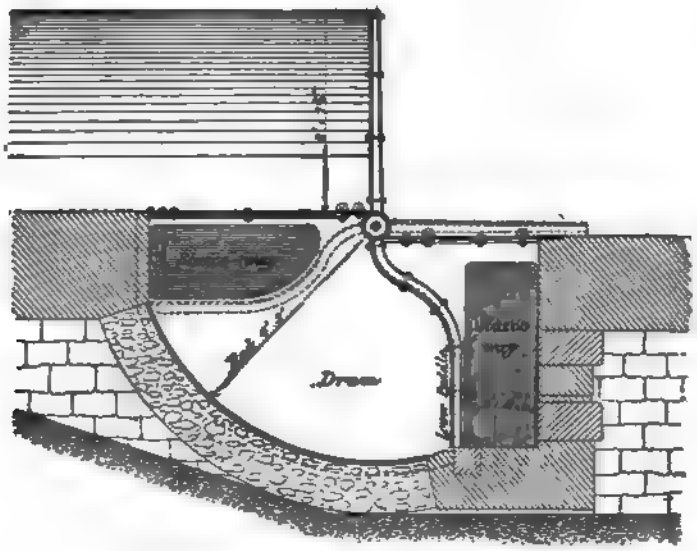
DRUM WEIR ON THE SPREE, GERMANY.



SECTION OF CHITTENDEN DRUM DAM ON THE OSAGE RIVER, MISSOURI. (1901.)

any movable dam and is not expensive, and had its advantages been better known it would doubtless have been more widely applied. It has been urged against it that the masonry and recess were expensive, but as it is only applicable to weirs which

in any case should have masonry foundation, and as the expense of constructing the recess would probably be offset by the cube of masonry saved, the objection would not often apply. The greatest objection to its application in its original form to American rivers would be that the reaction of the overfall would carry débris under the down-stream side of the wicket, preventing its proper bedding when lowered. This might be overcome by the use of a sliding apron hinged to the top of the wicket, or by sloping the masonry sharply away from the hinge on the down-stream side.



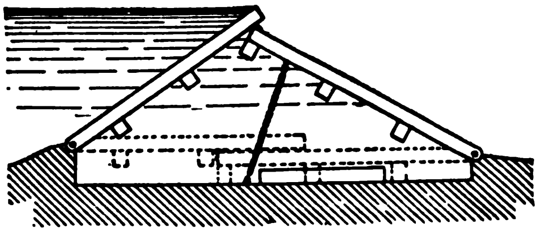
SECTION OF DESFONTAINES DRUM DAM, RIVER MARNE. (1867.)

In the Osage River type, as designed by Captain H. M. Chittenden, Corps of Engineers, this objection has been removed, as the drum is shaped like a sector of a circle, allowing no place for the lodgment of drift. This is gained, however, at some expense of facility of maneuvers. In the Desfontaines drum the weight of the arms is practically balanced on the hinge, thus requiring only a small head to move them. In the other the entire weight of the wicket has to be raised. It is necessary also to arrange for the admission of water inside it so that it will not float up during floods, and at the same time to arrange for the escape of this weight when it is desired to raise the dam.

BEAR-TRAP DAMS.

General.—The bear-trap was the pioneer of movable dams, although it was practically unknown until within recent years. The first one was built by White & Hazard in 1818 on the Lehigh River in the United States, and in the year following twelve more were built. The type disappeared, however, for forty or fifty years, except for occasional examples on lumbering streams, the next one applied to navigation being that on the river Marne in France, where a gate was built 28 feet 8 inches long and 9½

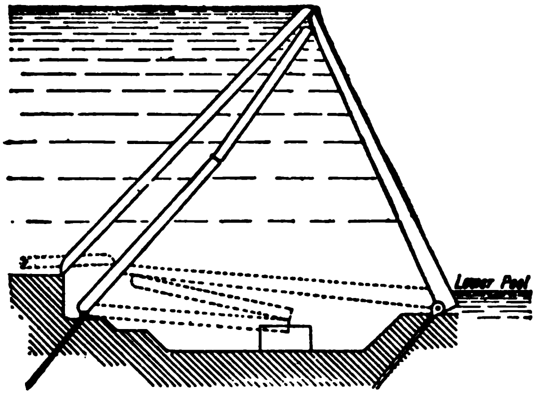
feet high above the sill. It was so badly proportioned that it proved a failure, after having cost $488 per lineal foot to construct, and as a result the type was generally condemned. In the last few years, however, other examples have been built in America of lengths from 14 feet to 120 feet and of heights from 7 feet to 14 feet, and have shown that with proper designing the bear-trap is a valuable device.



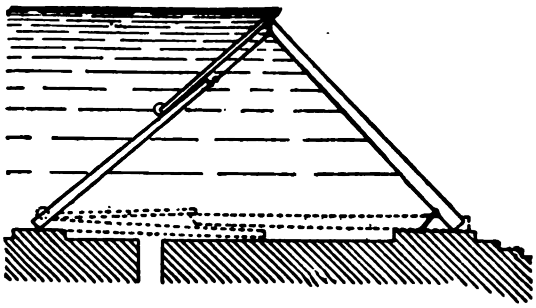
SECTION OF OLD BEAR-TRAP DAM.

Description.—The bear-trap, as usually built, consists of three varieties, the original or "old" type, the Parker, and the Lang. The first consists of two straight leaves, hinged at the bottom, the up-stream leaf overlapping the down-stream one when lowered, and when the water is introduced underneath it pushes up both, the end of the latter sliding along and helping to push up the former.

In the Parker type the up-stream leaf is divided into two parts, hinged together, so as to save width in the foundation, and the tops of both leaves are hinged together, thus avoiding the sliding friction of the old type. Sometimes the down-stream leaf is divided instead of the up-stream one, in which case the structure is known as the reversed Parker.



SECTION OF PARKER BEAR-TRAP DAM WITH IDLER LEAF.



SECTION OF LANG BEAR-TRAP DAM.

The Lang gate is the same as the Parker, except that the upper part of the up-stream leaf is replaced by a chain, and the opening covered by a sliding "idler" leaf.

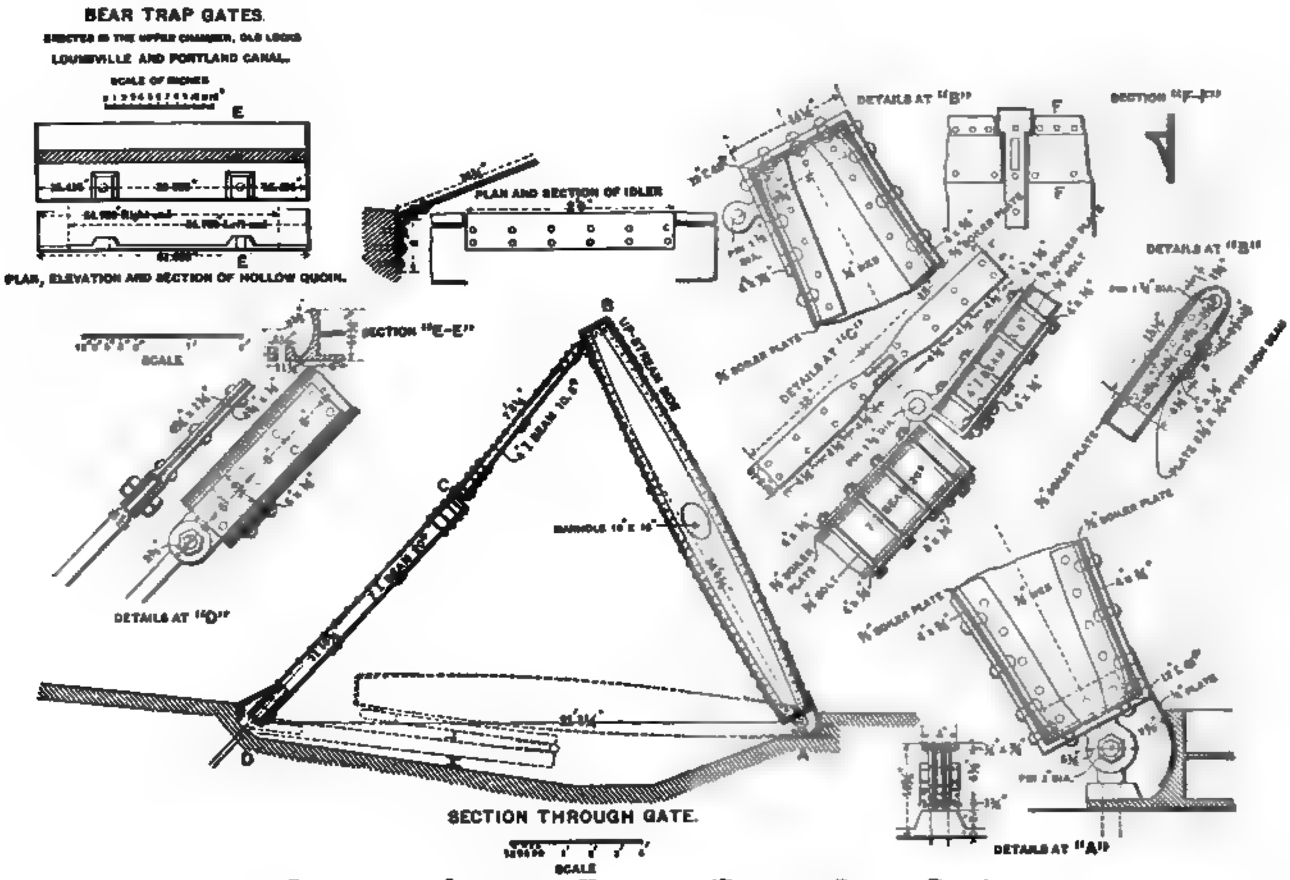
Many other varieties have been proposed, but, with the exception of one built for a regulating-weir on the Chicago Drainage Canal, none has come into use.

The following gates have been constructed by the United States and are still in operation. One or two others were also built, but were subsequently removed.

Parker Gates.—One on the Muscle Shoals Canal, Tennessee, built in 1892. Length, 40 feet; height, 8.5 feet.

One on the Louisville and Portland Canal, Kentucky, built in 1897. Length, 40 feet; height, 15 feet $3\frac{1}{2}$ inches; made of steel.

Lang Gates.—Sandy Lake Reservoir, Minnesota, built in 1895. One gate 11 feet long, 12 feet high; two gates used as lock-gates, each 40 feet long and 13 feet high, made of wood.



BEAR-TRAP AT LOUISVILLE, KY., 1896. (REVERSED PARKER TYPE.)

Old Bear-trap Gates.—Two of this type are under construction on the movable dams of the upper Ohio, each with a length of 120 feet and a height of 13 feet 2 inches.

Calculations.—To determine the dimensions of the old-style two-leaved and of the Parker bear-trap, the following data have been deduced* (Fig. 23):

OLD BEAR-TRAP.

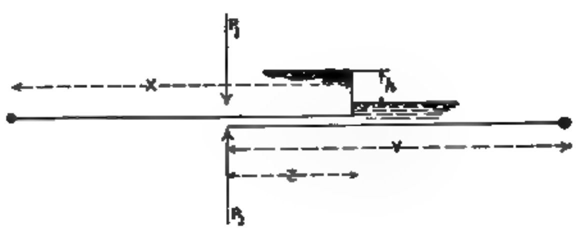


FIG. 23.

Let X be the length of the up-stream leaf;
Y the length of the down-stream leaf;
Z the overlap of the two leaves;
P_1 the downward pressure;
P_2 the upward pressure;
h the difference of level between the entry and the exit of the operating flume, and w the weight of a cubic foot of water.

* Journal of the Association of Engineering Societies, June, 1896, from a paper submitted by the Civil Engineers' Society of St. Louis, by Captain H. M. Chittenden, U. S. A., and Mr. Archibald O. Powell. The accompanying curves are reproduced by permission from the same paper.

Then $$P_1 = \frac{X.Z - \frac{1}{2}Z^2}{X - Z} . h . w, \qquad P_2 = \tfrac{1}{2} . Y . h . w.$$

If $P_1 = P_2$, $$Y = \frac{2X . Z - Z^2}{X - Z}.$$

For the gate to move, P_2 must be greater than P_1.

Let the relation be represented by $nP_2 = P_1$, n being a proper fraction. Then

$$Y = \frac{2X . Z - Z^2}{n(X - Z)}.$$

Combining this with an equation for the length of the gate, which is taken as unity, we find

$$Y = \frac{1 - \frac{n}{2}}{1 - n} - \sqrt{\frac{X}{1 - n} + \frac{1}{4}\left(\frac{n}{1 - n}\right)^2},$$

$$X = \sqrt{(1 - n)Y^2 - 2\left(1 - \frac{n}{2}\right) Y + 1.}$$

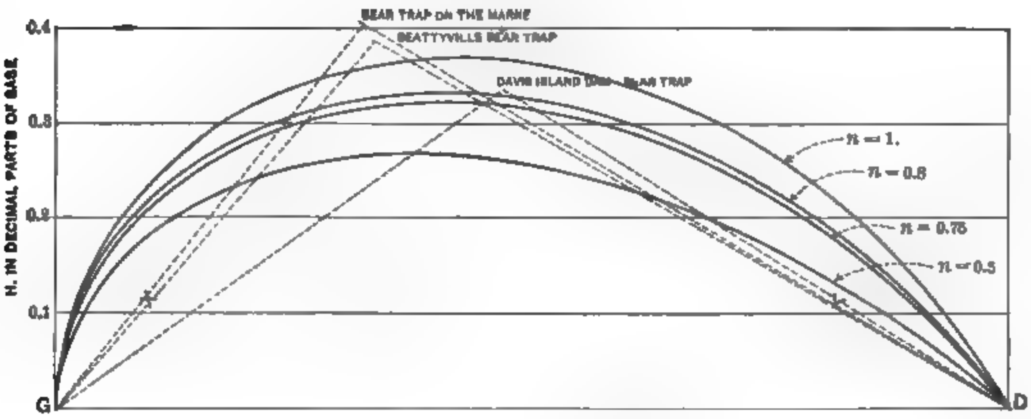


FIG. 24.—OLD BEAR-TRAP CURVES.

By substituting values of n between zero and 1 we find the corresponding values of X and Y. From this was platted the accompanying curve (Fig. 24) showing the apexes of the ordinary bear-trap when at full height. From it the lengths can be scaled, and it will be noted that the range is very wide. To secure safe results, however, it is best that n should not exceed $\frac{8}{10}$ or $\frac{4}{5}$.

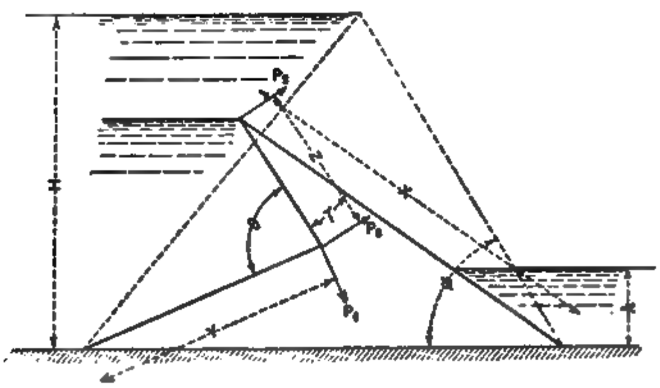


FIG. 25.

Parker Bear-trap.—The general solution for this type is given by the equation

$$P_4 - P_3 . \cos\theta - P_2 \cos\gamma . \sin\theta = 0,$$

the nomenclature being as shown in Fig. 25. From this as a basis the accompanying tables were deduced, and the curve of proportions platted (Fig. 26), the analysis

being similar for the direct, or up-stream, folding gate, and for the reversed, or down-stream, folding gate.

TABLE OF PARKER-GATE PROPORTIONS.

ϕ	No Backwater				$\frac{h}{H}=0.8.$				$\frac{h}{H}=0.6.$			
	X	Y	Z	[illegible]	X	Y	Z	H	X	Y	Z	H
10°	994	090	.084	173								
20°	.975	.185	.160	333								
30°	946	280	226	.473	......	......	......	......	855	.322	.177	.427
40°	.905	.376	281	582	.968	353	.321	.622	.784	.430	.214	.504
50°	854	470	324	654	896	.456	.352	686	.702	.533	.235	.537
60°	793	560	.353	687	800	558	.358	.693	.615	.630	245	533
70°	.724	645	369	680	677	.660	.337	.636	.524	716	.240	.492
80°	.649	722	.371	639	.542	.754	.296	.534	.432	.793	.225	.425
90°	.568	791	.359	568	437	.827	264	.437	.350	.854	.204	.350

ϕ	$\frac{h}{H}=0.4$				$\frac{h}{H}=0.2$				$\frac{h}{H}$ = limit.			
	X	Y	Z	H	X	Y	[illegible]	H	X	Y	Z	H
10°	......	......	......	......	......	......	......	......	.9129	.137	.050	.159
20°	......	......	......	......	......	......	......	......	8264	267	.093	.283
30°	.782	362	.144	391	748	.383	.131	.374	.7412	388	128	370
40°	.704	471	.175	453	.666	492	.158	.428	6580	.497	.155	423
50°	622	.572	.194	.476	587	.590	177	450	5773	.59	.173	.442
60°	.538	664	.202	.466	.509	678	.187	441	.5000	683	.183	433
70°	458	744	.202	.430	.433	.755	.188	407	.4264	758	185	400
80°	.386	810	196	380	362	.820	.182	.357	3572	822	179	352
90°	.322	864	186	.322	.302	.871	.173	.302	2929	874	167	293

Lang Bear-trap.—The following is a table of data for proportioning a bear-trap of the Lang type, the letters and figure being the same as for the Parker gate *

TABLE OF LANG-GATE PROPORTIONS.

ϕ	No Backwater				$\frac{h}{H}=0.2.$				$\frac{h}{H}=0.3$			
	X	Y	Z	H	X	Y	[illegible]	H	X	Y	Z	H
$42\frac{1}{2}$°	......	......	......	......	......	......	......	......	706	.485	191	478
45	......	......	......	......	......	......	......	......	691	.508	190	.490
$47\frac{1}{2}$	......	......	......	......	.715	512	227	528	.670	.534	.204	494
50	729	521	.250	.558	697	.536	.233	.534	644	562	.205	493
$52\frac{1}{2}$	.714	.544	.258	.567	.668	564	232	.530	612	.591	.203	486
55	.672	.576	.249	.550	.637	.592	229	.522	584	618	.202	479
$57\frac{1}{2}$	.635	.608	.242	536	.616	.616	232	.520	559	.642	201	472
60	.611	.630	241	529	590	.640	.230	.512	.536	665	.201	463
$62\frac{1}{2}$	.585	.655	.240	.519	564	.664	.228	.500				

ϕ	$\frac{h}{H}=0.4.$				$\frac{h}{H}=0.6$				$\frac{h}{H}$ = .75 to 1.0.			
	X	Y	Z	H	X	Y	Z	H	X	Y	Z	[illegible]
$42\frac{1}{2}$°	.678	.499	.177	.457	.660	510	.170	.446	.676	.501	.177	.456
45	.651	529	.180	.461	632	.539	.171	.447	.653	.528	.181	.462
$47\frac{1}{2}$	.627	.556	.183	.462	.610	.565	.175	.451	.632	.553	.185	466
50	600	584	.184	.461	.590	.589	.179	.453	.611	.578	.189	468
$52\frac{1}{2}$	576	.609	.185	.457	.570	.612	.182	453	590	.602	.192	.469
55	551	634	185	.452	.551	634	.185	.451	.570	.625	.195	.467
$57\frac{1}{2}$	.527	658	185	.444	.529	.657	.186	.446	.549	.647	196	.463
60	.505	680	.185	.438	.510	.678	.188	.442	.529	.669	.198	.458
$62\frac{1}{2}$												

* Transactions American Socety of Civil Engineers, June, 1898. Tables calculated by Mr. T. C. Thomas.

Remarks.—The bear-trap is the only type of standard dam in regard to which there is little experience available. Those built in recent years have not been always successful, as the theory and the practice were alike undeveloped. At present, however, they are being more widely studied and adopted, two having been recently constructed with lengths of 120 feet and heights of 13 feet on the dams of the Ohio River, for use as drift-chutes and regulating-weirs.

The old bear-trap required a great width of foundation, which was a source of expense, but with the modified forms this objection has been reduced. The type is not as simple, nor is it as certain in operation, as it has usually been designed, as the

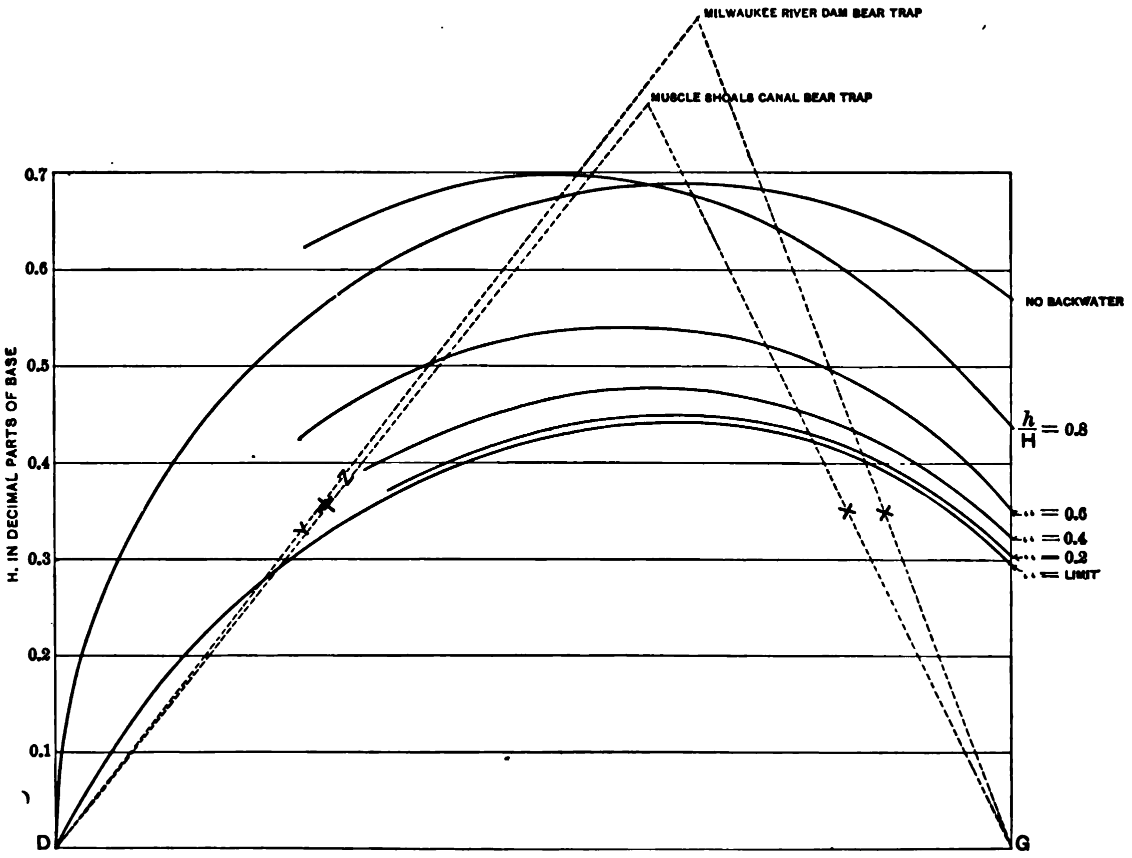


FIG. 26.—PARKER BEAR-TRAP CURVES.

drum wicket, but it can be applied to greater lifts, since it requires no substructure below it for a recess, as does the drum. It can therefore be used for passes where the latter would be inapplicable because of the expense of the foundation.

The head required to raise a gate should not be over 6 inches. On the Marne, owing to faulty proportions, a head of 2 feet was required, which was obtained by a row of Thénard shutters just above the gate. Where the head is not obtainable from the natural fall of the river it may be created by a pump and reservoir tank on shore.

One of the difficulties met with in long gates has been a tendency to warp, as the water has usually been introduced from one end only. This causes one end to rise

before the other, and twists and strains the leaves. Racks and pinions have been tried to reduce the tendency, but without much success. The most rational method of guarding against it would be to provide a culvert running the whole length of the foundation, so as to admit the water simultaneously through several openings, thus equalizing the pressure. It has also been suggested to use large separate pipes instead of a culvert and openings, each pipe being controlled by a valve on shore. By this means the pressure at any point could be regulated as desired. It has also been found desirable, for similar reasons, to form the hinges of long pieces, or shafts, instead of short pins, since with this construction the tendency to twist is more easily overcome.

CHAPTER X.

RECENT CONSTRUCTION OF MOVABLE DAMS.

In Bohemia.—Probably the most interesting work of this character at the present time is to be found in Bohemia, where a system of slackwater is being established between Aussig, on the Bohemian Elbe, and Prague, on its tributary, the Moldau, a distance of 190 miles.* When completed there will be thirteen locks, seven on the Elbe and six on the Moldau, and twelve movable dams closed with needles only, or with needles and Boulé gates combined. The lift of the highest lock at Troja, just below Prague, is given as 17.7 feet, and the smallest lift among the other locks will be 6.2 feet, the pool lengths varying from 2½ to 8 miles. The total estimated cost of the improvement is $4,320,000, or $22,800 per mile.

These rivers have always formed the chief natural arteries for the commerce of the kingdom, there being records of the existence of traffic upon them in the year A.D. 950. On the Elbe alone in 1822 there was a commerce of 19,700 tons, while in 1890 it had risen to more than three million tons. The usual processes of regularization had been employed, but these gradually became insufficient to meet the demands of traffic, and in 1892 it was recommended that a system of slackwater be established, and, after the necessary surveys had been made, the project was finally approved by the Government in December, 1895. The rivers flow through a hilly country and are subject to sudden rises and to much ice, and as the valleys are thickly settled and manufacturing and agricultural interests very important, it was evident that only movable dams would be suitable.

The largest boats navigating the rivers are 230 feet long, 36 feet wide, and draw 6 feet on a burden of 700 tons. The minimum depth of water to be secured was therefore fixed at seven feet.

The locks, wherever possible, will be placed in derivations, and will be all connected by telephone or telegraph.

The project also comprises the creation of two harbors of refuge for protecting craft from floods or ice, and they will be capable of sheltering 180 large boats. Fishways are to be provided at all dams.

At the present time (1902) the first four dams, those of Troja, Klecan, Libschitz,

* The information, together with the data for the drawings of certain details of construction, is obtained from "La Canalisation de la Moldau et de l'Elbe en Bohême." V. Rubin, Prague, 1900.

View of the Raft Chute during Construction at the Troja Dam.

[a] (To face p. 269.)

View of the Raft Chute at the Troja Dam, Bohemia, Showing the Passage of a Raft 11 Meters Wide.

(b) (To face p. [illegible])

and Mirowitz, are in operation. A brief description of them is given in the following paragraphs.

Dam No. 1 at Troja.—This dam lies just below Prague, and will provide the last pool of the system. It was commenced in 1899 and consists of three openings, two of 122 feet and one of 153 feet in width, closed with needles from 12.2 feet to 15.3 feet long, supported by fixed bars. The general construction and details of the dam and of the lock are similar to those at Klecan. The lift of the lock is given as 17.7 feet. A chute is provided for rafts 39 feet wide and about 1400 feet long, with slopes varying from 1 in 200 to 1 in 100.

Dam No. 2 at Klecan.—Work was commenced here early in 1897. The level of the pool above the original normal stage of water was to have been 11.8 feet, but as this was considered too great for needles, it was changed to 10.2 feet. The lift of the lock is given as 10 feet. The openings are three in number, two weirs of 127 feet each, and a pass of 131 feet, built chiefly with granite foundations and with wide aprons to guard against undermining. They are all closed with needles supported by swinging escape-bars on the Kummer system.

The trestles are spaced 4 feet 1 inch apart and connected to each other by fixed chains, and are 12.1 feet, 13.5 feet, and 15.4 feet high over all. They are protected when down by a sill 2 feet high. The aprons or floors are of iron and hinged to the trestles. When lowering, a small crab is taken out on the dam and attached to each trestle-head in turn, and the preceding trestle is lowered by connecting its chain with the crab. Another method is to place the crab on the pier and take a long chain out on the foot-bridge. The trestle-chains are then attached in turn to its end, and can thus all be lowered without moving the crab.

The needles are of larchwood, and provided with iron handles, hook-shaped. They are 10.8 feet, 12.0 feet, and 13.0 feet long, and measure 3.7 inches, 3.9 inches, and 4.7 inches square, and when wet weigh 46 lbs., 55 lbs., and 72 lbs., respectively. Thirteen are used to each bay, all connected to a rope.

The lock is of cut stone and rubble masonry combined, and is 722 feet long over all, divided by a pair of gates into two chambers, one about 220 feet long and 36 feet wide, the other about 460 feet long and 65½ feet wide. The width between the walls at each end is 36 feet, and one chamber wall is built on the same straight line from end to end. The filling or emptying of the two chambers as a whole takes 18 minutes. This is accomplished by means of culverts in the walls, closed by vertical wickets. The middle gates are provided with butterfly valves.

All the gates are composed of steel frames sheathed with wooden plank, and are operated by spars and racks placed under the coping. The upper gates are 9½ feet high, and the two lower pairs 19½ feet high. Their tops are all 16 inches above pool.

On the right bank a chute for rafts was built 39 feet in width and composed of a long and a short slope. As first constructed, the latter was made 1 in 10, and 134 feet long; this, however, was found to be too steep, and it was changed to a mean slope

of 1 in 24 and provided with six steps each 5 inches high, with slopes between of 1 in 40. The inflowing water, which was 4.6 feet deep at its commencement, was reduced by velocity to a depth of 3.3 feet at the outlet. The chute is closed by a needle dam.

The lock and dam were opened to navigation Feb. 22, 1899.

Dam No. 3 at Libschitz.—In this dam, where the lift is 12.7 feet, there are two openings, a weir 160 feet long, with 10.4 feet on the sill, and a pass 213 feet long, with 14.7 feet on the sill. The weir is closed by means of needles 12½ feet long provided with hooks which go over the escape-bars, as at the dam at Joinville-sur-Marne. They are supported by trestles 4 feet apart, 13.7 feet high, and weighing with all attachments 1400 lbs. each. These trestles are formed of Mannesmann tubes, protected when down by a sill 1 foot 6 inches high, the lowering being accomplished in the same manner as at the Klecan dam.

The trestles, of both the pass and the weir, are lowered towards the center pier, which is composed of cut stone and is 19½ feet wide, with its coping 20 inches above pool.

The pass is closed by wooden Boulé gates, supported by trestles formed of channel-irons. Five gates are used to each bay, four lower ones, each 3¼ feet high, and one top one, 1 foot 7 inches high. The lowest gate is 5 inches thick. They are put in place and maneuvered by a small traveling crane.

The trestles are 4 feet 1 inch apart and 19.7 feet high, and weigh 2800 lbs. each, or with all attachments, 3740 lbs. each.

The rails for the service car are hinged to the top members of the trestles and carry sheet-iron floors between, and form the connection between the members. The recess of the sill is 3 feet 3 inches deep.

The raising or lowering of the trestles is accomplished by means of a crab fixed on the bank, operating a long steel chain which passes above the trestles and to which each one is attached at the middle of its top member by a cast screw-clamp when its turn comes for lowering. This clamp is bolted to the trestle and forms the surface over which the chain slides, no sheave being used. The trestles are clamped 8 feet apart on the chain and can be raised six at a time. They are 7 feet 4 inches wide on top and provided with a hand-rail, and two widths of floor are used, one on each side of the chain. By this means a double track is provided for the service cars.

The lock and gates are similar to those at Klecan, but the filling-wickets slide horizontally and are mounted on a truck with six wheels, to avoid sliding friction, after the manner of a Stoney gate. The emptying wickets, which lift up, are similarly mounted. All are worked by hand.

A raft chute is provided of construction similar to that at Klecan, except that it is closed by a segmental, counterbalanced drum shutter, like a Taintor gate, lowered into a recess in the floor. This chute is 324 feet long, commencing with a four per cent grade, reduced to one per cent, and is provided every 20 feet with steps 5 inches in height. The work was completed in 1900.

View of Trestles of Boulé Dam when Lying Down, Libschitz Dam, Bohemia, Showing the Double Footway and Track Taken during Construction.

(To face p. 270.)

VIEW OF THE RAFT CHUTE AT THE LIBSCHITZ DAM, WITH A RAFT PASSING THROUGH.

(To face p. 371.)

Dam No. 4 at Mirowitz.—This was commenced in 1900. It consists of two needle weirs and of a pass 184 feet wide closed by Boulé gates. The needles are supported by trestles, and the gates by frames lowered from a bridge and hinged to its downstream side, and arranged so that they can be hoisted above the highest floods. These frames are 33 feet long and spaced in pairs about 6½ feet apart, and carry 16½ feet on the sill. The gates are of iron buckled-plates, each supported on a frame with rollers.

This arrangement of the dam was adopted, as it was desired to provide a highway bridge across the river at that point.

In America.—The largest system of movable dams in America, and one which, when ultimately completed, will compose the largest system in any country, is that of the Ohio River. At present only one dam is actually in operation, that at Davis Island, a few miles below Pittsburg, at the head of the river. This was the pioneer of movable dams in this country, having been completed more than twenty years ago in the face of strenuous opposition from the navigation interests, who believed that any structure in the river would seriously hamper traffic. One result of its construction has been that these interests have since come to see that a suitable system of slackwater is of the greatest benefit to commerce. Several other dams, Nos. 2, 3, 4, and 5, are under construction and will be completed in a few years. All of these are to be of the Chanoine type, with drift-chutes and regulating-weirs of bear-traps. Dam No. 6, near Beaver, Pa., which was commenced in 1892, is to be completed in 1903. It consists of a pass 600 feet long, closed by Chanoine wickets having a vertical height above the sill of 13 feet 2 inches; two weirs of the old bear-trap type each 120 feet long; and an A-frame weir also 120 feet long. The weirs have the same depth on their sills as is provided on the sill of the pass. The lock chamber is 600 feet long and 110 feet wide, closed by rolling gates. In addition to these dams several others will be commenced before long, the project being to build dams below large cities and the mouths of navigable tributaries first, and later to construct those between.

Several of the tributaries of this river have already systems or parts of systems of slackwater navigation. At its head is the Monongahela River with an old system of nine locks with fixed dams, and six more being constructed by the United States. All the new ones will be of concrete, including the dams. On the Allegheny River, the other head tributary, a lock and Chanoine movable dam are being built, and two others with fixed dams proposed. Farther down, in the State of Ohio, is the Muskingum River, with eleven locks and fixed dams, some of which are old, others rebuilt by the United States. In West Virginia are the Little Kanawha River with four locks and fixed dams belonging to a private corporation and one belonging to the Government, and the Kanawha River with ten locks, two fixed dams and eight Chanoine dams. The Big Sandy River follows with its lock and needle dam, and two others nearing completion, as described in the Appendix. The Kentucky River has a system of nine locks with fixed dams, and others in process of construction; and Green

River with its tributaries, Rough and Barren Rivers, has seven locks with fixed dams. The Wabash River, the largest northern tributary, has one lock with a fixed dam, while on the Tennessee and Cumberland Rivers, the largest southern tributaries, other locks are under construction.

When a project for the entire river has been completed it will comprise a system from Pittsburg to Cairo, over a thousand miles on the main artery alone; and with a valley as rich as that of the Ohio, the resulting development of commerce will be enormous.

With the exception of this river and its tributaries, there are at present few movable dams projected or under construction, the most noteworthy exception being that of the drum dam on the Osage River.

APPENDIX A.

In the following tables will be found the dimensions, etc., of locks and dams on certain rivers and canals in the United States.

ALLEGHENY RIVER, PA.

LOCKS.			
Location	Herr's Island	Six-mile Isl'd	Springdale
Reference number	1	2	3
Distance from junction with Ohio River, miles	1.8	6.9	16.9
Height of pool above sea, feet	710 0	721.0	733.0
Underlying material	Gravel	Rock	Rock
Completed	1893–1897	Put under contract in 1897	
Available length, feet	286.2	289.6	289.6
Clear width, feet	55.0	56.0	56.0
Lift, feet	7.0	11.0	12.0
Depth on upper miter-sill, feet	13.0	8 0	8.0
Depth on lower miter-sill, feet	8 0	7.0	7.0
Material	(A) concrete	Concrete	Concrete
River walls:			
Height above lower sill, feet	20.0	26.0	27.0
Height above floor, feet	(B) 40.0 and 22.0	28.0	29.0
Bottom width, feet	9.0	12.1 and 17.2	12.7 and 18.3
Top width, feet	9.0	12.1 and 7.0	12.7 and 7.0
Gates:			
Material	Wood	Wood	Wood
Kind	Miter	Miter	Miter
How operated	Hand	Hand	Hand
How filled and emptied	(C) Culverts	(D) Culverts	(D) Culverts
DAMS.			
Type (movable or fixed)	Movable	Fixed	Fixed
Underlying material	Gravel	Gravel	Gravel
Material of dam	Foundation concrete	Timber cribs filled with rip-rap.	Timber cribs filled with rip-rap.
Length from lock to abutment, feet	638.0	1230.0	923.0
Base width, feet	(E) 75.5	50.0	50.0
Weirs:			
Number and length	2 { 184′, 184′		
Closure (wickets, needles, etc.)	Chanoine wickets		
Level of sill to lower pool, feet	4.0 and 2.0		
Worked from bridge or boat	Bridge		
Navigation pass:			
Length, feet	250.0		
Closure	Chanoine wickets		
Level of sill to lower pool, feet	6.0		
Worked from	Bridge		
COST.			
New locks and dams complete, estimate	$1,132,000	for the three	

(A) Upper and lower ends of both walls built of dimension Cleveland sandstone.
(B) Lock has double floor; concrete at bottom of foundation and wooden floor above.
(C) Culverts around quoins and six in each side wall; all closed by circular damper-valves.
(D) Filled by six culverts in miter-wall, from cross-culvert, fed by two square damper-valves in upper buttress, emptied by culverts around lower quoins; damper-valves.
(E) Foundation of navigable pass.

BIG SANDY RIVER, W. VA. AND KY.

LOCKS.			
Location	Catlettsburg	Kavanaugh	Louisa
Reference number	1	2	3
Distance from junction with Ohio River, miles	½	13	26½
Pool above sea-level, feet	512.0	524.6	536.6
Completed	Building	in 1902	1896
Available length, feet	158	158	158
Clear width, feet	55	55	52
Lift, feet	15.2 (Max. 18.0)	12.6 (Max. 18.0)	12
Depth on upper miter-sill, feet	22.5	18.6	11.7
Depth on lower miter-sill, feet	7.3	6.0	6
Built of	Concrete	Concrete	Stone
River walls:			
Height above lower miter-sill, feet	27	23	21.5
Height above upper miter-sill, feet	27	23	19
Bottom width, feet	17.8	21	15
Top width, feet	5.2	5.2	7.5
Gates:			
Operated by	Hand	Hand	Hand
Material	Steel	Steel	Wood
Type of filling-valves	4 cylindrical in walls	4 cylindrical in walls	8 balanced in gates
Type of discharge-valves	4 cylindrical in walls	4 cylindrical in walls	2 balanced in walls
DAMS.			
Type	Needles and steel wickets*	Needles and steel wickets*	Movable (needles)
Built on	Rock	Rock	Rock
Built of	Concrete	Concrete	Stone and concrete
Net length, feet	300	276	270
Width at base, feet	22 to 34	23 to 36	24
Cost of lock, buildings, etc., about	$125,000	$143,000	$130,000
Cost of dam, about	....	....	$70,000
Length of pass, feet	140	140	130
Length of weir, feet	160	136	140

Operating Force.—The employees at the Louisa lock are three in number, with wages ranging from $45 to $55 per month.

* As proposed, 1902.

BLACK WARRIOR RIVER, ALABAMA.

LOCKS.			
Location	Tuscaloosa, Ala.	Tuscaloosa, Ala.	Tuscaloosa, Ala.
Reference number	1	2	3
Miles from Mobile	361.9	362.3	363.1
Pool above sea-level, feet	100.86	109.36	119.86
Built on	Sandstone	Sandstone	Sandstone
Built of	Cut sandstone	Cut sandstone	Cut sandstone
Begun	1888	1890	1892
Completed	1894	1895	1895
Quantity masonry, cubic yards	9762	10728	10885
Extreme length, feet	393	390½	390½
Available length, feet	286	286	286
Clear width, feet	52	52	52
Lift, feet	10	8½	10½
Depth on upper miter-sill, feet	9	7½	9
Depth on lower miter-sill, feet	6½	6½	6½
Wall height above lower miter-sill, feet	29½	30½	32
Wall height above floor, feet	30	31	32½
Material of floor	Natural rock	Natural rock	Natural rock
Chamber-wall width at floor line, feet	14	14	12
Chamber-wall width at coping, feet	7	7	7
Type of gates	Miter	Miter	Miter
Material of gates	Steel	Steel	Steel
Operating power	Hand	Hand	Hand
Location of filling-valves	Wall	Wall	Wall
Location of emptying-valves	Wall	Wall	Wall
Total gross area of filling-valves, square feet	56	56	60
Total gross area of emptying-valves, sq. ft.	58	58	60
Type of filling-valves	Balanced	Balanced	Balanced
Type of emptying-valves	Balanced	Balanced	Balanced
Type of head-bay coffer	Needles	Needles	Needles
Type of tail-bay coffer	None	None	None
Number of men employed	2	2	2
Wages per month	$45 and $35	$45 and $35	$45 and $25
DAMS.			
Type	Fixed	Fixed	Fixed
Step or slope	Step	Step	Step
Built on	Sandstone	Sandstone	Sandstone
Built of	Dry rubble masonry	Dry rubble masonry	Dry rubble masonry
Extreme length, feet	339	409	650
Extreme width of base, feet	21	24	26
COST.			
Lock and abutment	$221,078.90	$146,880.67	$131.561.47
Dam	10,878.62	10,902.25	16,336.83
Guide-cribs	1,276.98	909.35	1,390 50
Grounds	175.00	125.00	25.90
Buildings	2,000.00	1,700.00	1,700.00
Miscellaneous	6,915.00	6,005.00	5,858.00
Total	$242.324.50	$166,502.27	$156,871.80

FOX RIVER, WISCONSIN.

LOCKS.									
Location	Portage	Fort Winnebago	Governor Bend	Montello	Grand River	Princeton	White River	Berlin	Eureka
Reference number	27	26	25	24	23	22	21	20	19
Built on	Sand	Clay	Sand	Sand	Clay	Sand	Clay	Clay	Piles in sand
Built of	(a)	(b)	(c)	(a)	(d)	(d)	(d)	(d)	(d)
Last rebuilt or repaired	1901	1901	1900	1901	1875–8	1875–8	1875–8	1875–8	1875–7
Extreme length, feet	215.7	200.7	203.8	200.0	225.6	220.0	226.2	220.6	220.25
Length between hollow quoins, feet	165.3	160.2	160.4	160.5	170.3	170.4	170.5	170.6	170.65
Width between chamber walls at top, ft.	35.1	34.3	35.0	35.3	34.7	34.9	34.5	34.8	35.7
Face of chamber walls, plumb or batter per foot	Plumb	Plumb	Plumb	Plumb	Plumb	Plumb	Plumb	Plumb	Plumb
Lift at mean stage, feet	(9.00 max.)* 2.2	5.9	3.1	3.1	1.1	0.9	1.4	2.1	2.7
Depth on upper sill at mean stage, feet	7.31	10.23	7.14	7.81	6.75	8.10	7.33	8.20	8.63
Depth on lower sill at mean stage, feet	5.09	5.89	5.58	6.07	10.0	9.95	10.87	8.87	10.02
Height of walls above floor, feet	18.4	14.0	14.5	15.0	16.0	14.55	16.4	14.35	15.3
Width of walls on chamber coping, feet	7.2	4.4	6.6	6.2	4.0	4.0	4.8	3.9	4.6
Number and location of filling-valves	4 gate	6 gate	4 gate	4 gate	4 gate	4 gate	4 gate	4 gate	4 gate
Number and location of emptying-valves	6 gate	6 gate	4 gate	4 gate	6 gate	6 gate	6 gate	6 gate	6 gate
DAMS.									
Lift	6.52	2.60	5.72	5.16	2.66	4.00	5.20	3.27	2.65
Built of	Canal connecting Wisconsin and Fox rivers, Portage to Ft. Winnebago lock	Old scow, sunk to arrest sedim't	Timber crib filled with stone	Timber crib filled with stone	Timber crib filled with stone	Timber crib filled with stone	Timber crib filled with stone	Timber crib filled with stone	Timber on piles
Extreme length, feet	....	about 60.0	59.6	176.6	120.0	180.0	200.0	200.0	209.0
Extreme width, feet	....	....	30.0	24.0	20.0	30.0	30.0	30.0	24.0
Height, feet	....	....	5.6	5.6	12.9	12.7	12.7	12.5	8.0
Length of pool below lock, miles	2.17	4.12	24.11	3.28	20.70	9.40	9.99	8.11	43.22

(a) Denotes upper end cut stone masonry, balance dry rubble and plank.
(b) Denotes upper and lower ends cut stone masonry, balance dry rubble and plank.
(c) Denotes all dry rubble, faced with plank.
(d) Denotes all cut stone masonry.

Operating Force.—At most of the locks one lock-tender is employed, but where the locks are close together, as at Appleton or Kaukauna, one man attends to two locks. The wages range from $25 to $35 per month.

* The maximum lift at Portage lock = height of coping above mean stage in canal.

FOX RIVER, WISCONSIN.—*Continued.*

LOCKS.									
Location	Menasha	Appleton First	Appleton Second	Appleton Third	Appleton Fourth	Cedars	Little Chute First	Little Chute Second	Little Chute Combined Upper
Reference number	18	17	16	15	14	13	12	11	10
Built on	Clay	Rock	Clay	Rock	Rock	Rock	Rock	Rock	Hard-pan
Built of	(b)	(d)	(d)	(d)	(b)	(d)	(a)	(d)	(d)
Last rebuilt or repaired	1899	1883–4	1900–1	1899–1900	1894–5	1887	1893–4	1881	1878
Extreme length, feet	211.35	220.8	219.6	212.0	206.4	220.4	208.3	220.3	199.2
Length between hollow quoins, feet	170.0	170.7	170.6	170.0	160.2	170.05	160.2	170.2	170.15
Width between chamber walls at top, ft.	35.4	35.0	34.8	35.0	35.0	35.0	35.4	35.0	36.35
Face of chamber walls, plumb or batter per foot	3/16" per foot	Plumb	Plumb	Plumb	1/4" per foot	Plumb	1/2" per foot (not used)	Plumb	3/8" per foot
Lift at mean stage, feet	9.2	10.1	9.7	8.5	8.2	9.8	7.15	14.5	10.8
Depth on upper sill at mean stage, feet	6.4	6.69	7.11	6.20	14.27	6.86	11.60	12.28	8.06
Depth on lower sill at mean stage, feet	6.19	6.0	6.0	8.72	7.70	6.85		6.0	6.0
Height of walls above floor, feet	20.5	21.5	19.8	20.80	19.2	20.5	14.7	26.30	18.8
Width of walls on chamber coping, feet	6.2	4.5	4.5	4.5	6.1	4.5	6.0	4.5	4.0
Number and location of filling-valves	6 platform	6 side valve	6 platform	6 platform	6 gate	6 side valve	4 gate	6 side valve	6 platform
Number and location of emptying-valves	6 gate	6 gate	6 gate	6 gate	6 gate	6 gate with hand-wheels	4 gate	6 gate with hand-wheels	6 platform
DAMS.									
Lift	9.68	29.14*	No dam	No dam	7.60	9.78	36.12*	No dam	No dam
Built of	Timber crib filled with stone	Large stone masonry	....	....	Timber crib filled with stone	Timber crib filled with stone	Timber crib filled with stone	....	....
Extreme length, feet	401.7	800.0	....	....	540.0	820.0	690.0	....	....
Extreme width, feet	24.0	13.0	....	....	24.0	24.0	24.0	....	....
Height, feet	11.0	9.0	....	....	8.3	12.0	13.0	....	....
Length of pool below lock, miles	5.71	1.26†	....	....	3.55	0.78	2.43†	....	....

(a) Denotes upper end cut stone masonry, balance dry rubble and plank.
(b) Denotes upper and lower ends cut stone masonry, balance dry rubble and plank.
(c) Denotes all dry rubble, faced with plank.
(d) Denotes all cut stone masonry.
* Total lift of the flight of locks.
† Total length of pool between the first and the last lock.

FOX RIVER, WISCONSIN.—*Continued.*

LOCKS.									
Location	Little Chute Combined Lower	Kaukauna First	Kaukauna Second	Kaukauna Third	Kaukauna Fourth	Kaukauna Fifth	Rapide Croche	Little Kaukauna	Depere
Reference number	9	8	7	6	5	4	3	2	1
Built on	Rock	Rock	Rock	Rock	Rock	Rock	Soft muck	Soft clay	Rock
Built of	(*d*)	(*d*)	(*d*)	(*d*)	(*d*)	(*b*)	(*d*)	(*b*)	(*b*)
Last rebuilt or repaired	1878	1883	1902–3	1878	1878	1897–8	1857–9	1895–6	1896–7
Extreme length, feet	186.7	200.9	223.0	220.8	220.1	227.5	201.6	200.5	225.8
Length between hollow quoins, feet	172.55	170.4	170.0	170.0	170.1	170.0	160.4	160.55	170.0
Width between chamber walls at top, ft.	36.45	35.05	35.0	36.6	36.6	35.6	36.6	36.6	35.83
Face of chamber walls, plumb or batter per foot	3/8" per foot	Plumb	Plumb	1/2" per foot	1/2" per foot	3/16" per foot	1/2" per foot	3/8" per foot	1/2" per foot
Lift at mean stage, feet	11.0	9.9	10.2	10.4	10.0	8.6	8.0	7.1	9.0
Depth on upper sill at mean stage, feet	.....	6.47	6.0	6.69	6.14	6.39	14.53	12.48	10.10
Depth on lower sill at mean stage, feet	8.13	6.0	6.21	6.00	6.00	8.94	7 68	6.74	7.02
Height of walls above floor, feet	21.2	21.9	19.66	20.4	20.0	21.4	20.8	19.1	20.65
Width of walls on chamber coping, feet	4.0	4.5	4.5	4.6	4.6	6.4	4.0	5.5	6.0
Number and location of filling-valves	6 platform	6 platform	6 platform	6 platform	6 platform	6 platform	6 gate	6 gate	6 platform
Number and location of emptying-valves	4 gate	6 gate	6 gate	6 gate	6 gate	6 gate	6 gate	6 gate	6 gate
DAMS.									
Lift	No dam	50.50*	No dam	No dam	No dam	No dam	9.42	6.17	8.31
Built of	....	Timber crib filled with stone	....	....	....	....	Timber crib filled with stone	Piles with stone filling and crib at toe of dam	Timber crib filled with stone
Extreme length, feet	....	582.0	....	....	....	....	578.0	588.7	987.8
Extreme width, feet	....	24.0	....	....	....	....	24.0	26.0	28.0
Height, feet	....	....	....	....	....	....	14.0	10.0	12.0
Length of pool below lock, miles	....	4.99†	....	....	....	....	6.08	5.82	....

(*b*) Denotes upper and lower ends cut stone masonry, balance dry rubble and plank.
(*d*) Denotes all cut stone masonry.
* Total lift of the flight of locks.
† Total length of pool between the first and the last lock.

GREEN, BARREN, AND ROUGH RIVERS, KENTUCKY.

(Barren and Rough rivers are tributaries of Green River.)

	Green River.					Barren River.	Rough River.
LOCKS.							
Location	Spottsville, Ky.	Rumsey, Ky.	Rochester, Ky.	Woodbury, Ky.	Glenmore, Ky.	Greencastle, Ky.	Livermore, Ky.
Reference number	1	2	3	4	5	1	1
Miles from junction with the River Ohio	8.5	62.8	108.5	149.5	167.5	15 from confluence	8 from confluence
Built on	Rock	Rock	Rock	Rock	Gravel	Rock and gravel	Rock
Built of	Masonry	Masonry	Masonry	Masonry	Concrete	Masonry	Concrete
Completed	1835–40	1835–40*	1835–40	1835–40	1899	1835–40	1896
Available length, feet	138	145	138	138	145	140	123
Clear width, feet	36	36	35.8	36	36	36	27
Lift	19.1 max.	15.4	15.4	16.7	14.0	15.1	9.0
Depth on upper miter-sill, feet	5.7	8.4	6.8	6.6	8.0	7.3	6.8
Depth on lower miter-sill	1.0 min	5.1	6.8	5.6	5.1	4.5	5.2
Height of wall above lower miter-sill, feet	32.0	30.0	32.1	30.6	29.0	29.0	21.5
Height of wall above foundation, feet	33.7	34.0	33.6	32.1	30.0	31.4	24.0
Material of floor	Rock	Rock	Rock	Rock	Timber	Timber	Rock
Width of wall at foundation, feet	9 and 10	16½ and 20	12	12	14 and 17	12	12
Width of wall at coping, feet	4½ and 8	6 and 12½	12	12	5 and 12	12	5 and 10
Type of gates	Miter	Miter	Miter	Miter	Miter	Miter	Miter
Material of gates	2 of wood, 2 of steel	Wood	2 of wood, 2 of steel	2 of wood, 2 of steel	Wood	2 of wood, 2 of steel	Wood
Operating power	Hand	Hand	Hand	Hand	Hand	Hand	Hand
Location of filling-valves	Gates	Culverts	Gates	Gates	Culverts	Gates	Gates
Location of emptying-valves	Gates	Culverts	Gates	Gates	Culverts	Gates	Gates
Total gross area of filling-valves, square feet	....	50	....	....	57	28	40
Total gross area of emptying-valves, square feet	....	50	....	....	57	28	40
Types of filling-valves	Gridiron	Balanced	Gridiron	Gridiron	Balanced	Gridiron	Balanced
Types of emptying valves	Balanced	Balanced	Balanced	Balanced	Balanced	Gridiron	Balanced
Types of head-bay coffer	Needles	Needles	Needles	Needles	Needles	Needles	Needles
Types of tail-bay coffer	Needles	Needles	Needles	Needles	Needles	Needles	Needles
Number of men employed	3	3	3	2	2	2	1
Wages per month	$50 and $45	$50 and $45	$50 and $45	$50 and $45	$50 and $45	$50 and $45	$50
Original cost	$179,110.00†	$95,700.00†	$121,380.00†	$125,720.00†	$173,320.00†	$135,730.00†	$72,190.02
DAMS.							
Type	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
Step or slope	Slope	Slope	Slope	Slope	Step	Slope	Step
Built on	Rock	Rock	Rock	Rock	Gravel	Rock	Rock
Built of			All of timber	cribs filled	with riprap.		
Extreme length, feet	528.5	598	353	381	282	268	185
Extreme width of base, feet	46	55	65	50	46	80	25.6

* Rebuilt in 1895-6, at a cost of about $170,000. † Includes cost of dam, etc.

NOTE.—Locks and dams 1 to 4, Green River, and lock and dam No. 1, Barren River, were built by the State of Kentucky, leased about 1865 to the Green River Navigation Company, and purchased by the United States in December, 1888.

KANAWHA RIVER, W. VA.

LOCKS.					
Location	Near Cannelton and Montgomery	Near Paint Creek	Near Coalburg	Near Marmet	4 miles below Charleston
Reference number	*2	3	4	5	6
Distance from mouth, miles	85¼	80	73¾	67¼	54½
Pool above sea, feet	597.75	587.42‡	573.75	566.50	559.00
Underlying material	Rock	Rock	Rock	Rock	Rock
Completed	1887	1882	1880	1880	1886
Available length, feet	271	272	274	274	313
Clear width, feet	50†	50	50	50	55
Lift, feet	10.33	13.67	7.25	7.50	8.50
Depth on upper miter-sill, feet	8.00	10.67	14.00	14.00	15.25
Depth on lower miter-sill, feet	8.67	7.00	6.75	6.50	6.75
Material	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone
River walls:					
Height above lower sill, feet	31	30 to 34.50	20	20	21.75
Height above floor, feet	38.50	37.50 to 42.00	26	24	28.50
Bottom width, feet	21	20	19	16.25	15.50
Top width, feet	5.25	5	5.33	5	5
Gates:					
Material	Timber	Timber	Timber	Iron	Timber
Kind	Miter	Miter	Miter	Miter	Miter
How operated	Hand	Hand	Hand	Hand	Hand
How filled and emptied	Filled by valves in floor of upper bay; emptied by valves in lower gates	Filled by valves in floor of upper bay; emptied by valves in lower gates	Gate-valves	Gate-valves	Gate-valves
DAMS.					
Type					
Underlying material	Fixed	Fixed	Movable	Movable	Movable
Material of dam	Rock	Rock	Rock	Rock	Rock
	Timber cribs filled with stone	Timber cribs filled with stone	Concrete, masonry, timber, iron, etc.	Concrete masonry, timber, iron, etc.	Concrete, masonry, timber, iron, etc.
Length from lock to abutment, feet					
Base width, feet	524	564.5	468	529	568
Weirs:	38	33			
Dimensions:					
Length, feet					
Width of foundation at bottom, ft.			210	265.5	310
Width of foundation at sill, feet			21	21	23
Average height of foundation, feet			20	18	21
Closure			13.65	13	11
			Chanoine wickets	Chanoine wickets	Chanoine wickets
Level of sill to lower pool (plus = sill lowest; minus = sill highest), feet					
Worked from bridge or boat			−1.25	−2.50	−1.50
Navigation pass:			Bridge	Bridge	Bridge
Dimensions:					
Length, feet					
Width of foundation at bottom, ft			248	250	248
Width of foundation at sill, feet			48.2	48	51
Average height of foundation, feet			48.2	48	50
Closure			6.75	5	8.3
			Chanoine wickets	Chanoine wickets	Chanoine wickets
Level of sill to lower pool, feet					
Worked from			5.5	5.5	4
			Bridge	Bridge	Bridge
COST.					
Lock and dam complete, estimate		$490,000	$275,000	$275,000	
Lock and dam complete, actual	$353,608				$337,625

* No. 1 is not included in present scheme of improvement.

† Original lift 12 feet; changed in 1895 by raising dam No. 3.

‡ Original pool level 585.75; feet; raised 1.67 feet in 1895.

KANAWHA RIVER, W. VA.—*Continued.*

LOCKS.					
Location	Near St. Albans (mouth of Coal River)	4 miles above Winfield and Red House	Near Fraziers Bottom	2½ miles below Buffalo	Near Pt. Pleasant
Reference number	7	8	9	10	11
Distance from mouth, miles	44¼	36	25¼	18¾	1¾
Pool above sea, feet	550.50	542.25	534.25	528.00	521.00
Underlying material	Rock and hard-pan	Rock	Rock	Rock	Hard-pan
Completed	1893	1893	1898	1898	1898
Available length, feet	313	313	313	313	313
Clear width, feet	55	55	55	55	55
Lift, feet	8.25	8.00	6.25	7.00	10.92
Depth on upper miter-sill, feet	15.00	16.25	13.75	14.00	7.00
Depth on lower miter-sill, feet	6.75	8.25	7.50	7.00	6.08
Material	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone
River walls:					
Height above lower sill, feet	20	21.25	19	19	22
Height above floor, feet	30.75	22.25	24.50	21.25	36
Bottom width, feet	18	13.75	16.08	13.42	23
Top width, feet	6	6	6.54	6.54	6.58
Gates:					
Material	Timber	Timber	Timber	Timber	Timber
Kind	Miter	Miter	Miter	Miter	Miter
How operated	Hand	Hand	Hand	Hand	Hand
How filled and emptied	Gate-valves	Gate-valves	Gate-valves	Gate-valves	Gate-valves
DAMS.					
Type	Movable	Movable	Movable	Movable	Movable
Underlying material	Rock and hard-pan	Rock	Rock	Rock	Hard-pan
Material of dam	Concrete, masonry, timber, iron, etc.	Concrete, masonry, timber, iron, etc.	Concrete, masonry, timber, iron, etc.	Concrete, masonry, timber, iron, etc.	Concrete, masonry, timber, iron, etc.
Length from lock to abutment, feet	574	550	542	542	678
Weirs:					
Dimensions:					
Length	316	292	284	284	364
Width of foundation at bottom, ft	27	25	26	24	46
Width of foundation at sill, feet	23	23	23	23	23
Average height of foundation, feet	14	8.17	9.95	7	21
Closure	Chanoine wickets	Chanoine wickets	Chanoine wickets	Chanoine wickets	Chanoine wickets
Level of sill to lower pool (plus = sill lowest; minus = sill highest), feet	+0.25	+0.50	+2.25	+1.50	−2.42
Worked from bridge or boat	Bridge	Bridge	Bridge	Bridge	Bridge
Navigation pass:					
Dimensions:					
Length, feet	248	248	248	248	304
Width of foundation at bottom, ft	56	51	51.5	51	60.7
Width of foundation at sill, feet	50	50	50	50	50
Average height of foundation, feet	12.8	4.25	5.65	4.17	18.5
Closure	Chanoine wickets	Chanoine wickets	Chanoine wickets	Chanoine wickets	Chanoine wickets
Level of sill to lower pool, feet	4.75	5	6.75	6	2.08
Worked from	Bridge	Bridge	Bridge	Bridge	Bridge
COST.					
New lock and dam complete, estimate	....	....	$315,000	$290,000	$650,000
New lock and dam complete, actual	$341,136	$281,975			

Operating Force.—At the locks with fixed dams four men are employed, with wages from $40 to $50 per month; at those with movable dams five men are employed, with wages from $35 to $55 per month.

LITTLE KANAWHA RIVER, W. VA.

LOCKS.					
Location	Shacktown	Leaches	Wells	Palestine	Burning Springs
Reference number	1	2	3	4	5
Distance from Ohio River, miles	3½	14	26½	32	40½
Pool above sea, feet	579.34	589.52	601.34	613.34	625.34
Underlying material	Rock	Rock	Rock	Rock	Gravel
Completed	1867	1867	1867	1867	1891
Available length, feet	125	125	125	125	126
Clear width, feet	22	22	22	22	26
Lift	15.73	10.18	11.82	12.00	12.00
Depth on upper miter-sill, feet	....	....	....	....	7.56
Depth on lower miter-sill, feet	3½	3½	3½	3½	4
Material	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone
River wall:					
Height above lower sill, feet	....	....	....	....	24.25
Height above floor, feet	....	....	....	....	25.7
Bottom width, feet	....	....	....	....	12
Top width, feet	....	....	....	....	6 and 12
Gates:					
Material	Wood	Wood	Wood	Wood	Wood
Kind	Miter	Miter	Miter	Miter	Miter
How operated	Hand	Hand	Hand	Hand	Hand
How filled and emptied	Gate-valves	Gate-valves	Gate-valves	Gate valves	Wall-valves
DAMS.					
Type	Fixed	Fixed	Fixed	Fixed	Fixed
Underlying material	Rock	Rock	Rock	Rock	Rock
Material of dam	Timber cribs filled with stone	Timber cribs filled with stone	Timber cribs filled with stone	Timber cribs filled with stone	Timber cribs filled with stone
Length from lock to abutment, feet	....	....	....	....	250
Base width, feet	....	....	....	....	50
COST.					
Lock and dam complete	$70,314	$60,623	$60,747	$58,680	$167,875

NOTE.—Nos. 1, 2, 3, and 4 belong to a corporation; No. 5 to the United States.

KENTUCKY RIVER, KENTUCKY.

LOCKS.								
Location	Carrollton	Lockport	Gest	Frankfort	Tyrone	Salvisa	High Bridge	Little Hickman
Reference number	1	2	3	4	5	6	7	8
Miles from Ohio River	4	31	42	65	82.2	96.2	117	138.2
Pool above sea-level, feet	429.57	443.82	456.67	470.87	485.12	499.12	514.60	532.60
Built on	Rock	Rock	Rock	Rock	Rock	Rock	Rock	Rock
Built of	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone	Cut stone
Completed	1841–44	1841–44	1841–44	1841–44	1844	1891	1897	1900
Extreme length, feet	235.5	236	235.6	235.7	235.7	249	255	245
Available length, feet	145	145	145	145	145	147	148	148
Clear width, feet	38	38	38	38	38	52	52	52
Lift, feet	16.57	14.25	12.85	14.20	14.25	14.0	15.48	18.0
Depth on upper miter-sill, feet	7.74	7.60	8.30	6.20	9.15	9.50	9.16	10.00
Depth on lower miter-sill, feet	6.00	5.84	6.50	6.10	6.50	6.00	6.70	6.00
Height of wall above lower miter-sill, feet	32.4	29.1	29.6	30.3	30.7	30.5	31.0	34.0
Height of wall above floor, feet	34.5	31.1	31.6	32.3	32.7	41.5	42.7	42.0
Material of floor	Rock	Rock	Rock	Rock	Rock	Rock	Rock	Rock
Width of wall at floor line, feet	15	15	15	15	15	18 and 23	18 and 23	20 and 23
Width of wall at coping	15	15	15	15	15	8 and 18	7 and 16	6 and 15
Type of gates. Material of gates. Operating power.	All gates are of the mitering type, of wood, and operated by hand.							
Location of filling-valves	Gates	Gates	Gates	Gates	Gates	Walls	Walls	Walls
Location of emptying-valves	Gates	Gates	Gates	Gates	Gates	Walls	Walls	Gates
Total gross area of filling-valves, sq. ft.	36	36	36	36	36	45	52	52
Total gross area of emptying-valves, sq. ft.	36	36	36	36	36	45	52	52
Type of filling-valves	Gridiron	Gridiron	Gridiron	Gridiron	Gridiron	Vertical balanced	Vertical balanced	Vertical balanced
Type of emptying-valves	Gridiron	Gridiron	Gridiron	Gridiron	Gridiron	Vertical balanced	Vertical balanced	Horizontal balanced
Type of head-bay coffer	None	None	None	None	None	Needles	Needles	Needles
Type of tail-bay coffer	None	None	None	None	None	Needles	Needles	Needles
Number of men employed and wages	Two men are employed at each lock at wages of $45 and $50 per month.							
Original cost, locks and dams and all other work complete	$211,300.00	$151,800.00	$135,700.00	$124,000.00	$127,100.00	$314,900.00	$290,800.00	$253,400.00
DAMS.								
Type	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
Step or slope	Slope	Slope	Step	Step	Step	Step	Step	Step
Built on	Gravel and rock	Gravel and rock	Gravel and rock	Gravel and rock	Rock	Rock	Rock	Rock
Built of	All dams are of timber cribs filled with riprap. Dam No. 1 has a coping of concrete.							
Extreme length, feet	428	432	464	528	550	412	350	about 240
Extreme width of base, feet	78	70	66	50	60	60	60	50
Cost	....	....	....	....	....	....	$24,900.00	

Note.—Locks and dams 1 to 5 inclusive were built by the State of Kentucky and transferred to the United States in 1880.

LOUISVILLE AND PORTLAND CANAL, KY.

Location	Louisville, Ky.
Number	Two locks, adjoining
Miles from Louisville	2½
Pool above sea-level	Zero of gauge, 398.5 feet
Built on	Limestone bed-rock
Built of	Sandstone and concrete
Completed	1873, by the United States; purchased from the Louisville and Portland Canal Co.; commenced in 1860
Extreme length, feet	744
Available length, feet	335.0 between miter-sills
Clear width, feet	80.0
Lift	Total, 24 feet; upper chamber, 14 feet; lower chamber, 10 feet
Depth on upper miter-sill, feet	Lowest stage, 6.73; highest, 17.68 *
Depth on lower miter-sill, feet	Lowest stage, 4.8
Height of wall above lower miter-sill, feet	26.24
Height of wall above floor, feet	28.24
Material of floor	Bed-rock
Width of wall at coping, feet	8
Type of gates	Miter, bowstring trusses
Material of gates	Oak and yellow pine
Operating power	Steam or hand
Location of filling-valves	Gates
Location of emptying-valves	Gates
Total gross area of filling-valves	9 sq. ft. each, 6 to each gate; total, 108 sq. ft.
Total gross area of emptying-valves	9 sq. ft. each, 6 to each gate; total, 108 sq. ft.
Type of filling-valves	Butterfly, vertical balanced
Type of emptying-valves	Butterfly, vertical balanced
Number of men employed	Operating force, 9 men, 2 shifts; total, 18 men per 24 hours
Wages of men employed	$1060.00 per month
Original cost	Not known

* Above this stage the lock is closed to navigation.

NOTE.—The dam is formed by the "Falls of the Ohio," the natural configuration of the river-bed.

MONONGAHELA RIVER, PA.

LOCKS.					
Location	Pittsburg	Port Perry	Elizabeth	North Charleroi	Brownsville
Reference number	1	2	3	4	5
Distance from junction with Ohio River, miles	1.0	12.0	25.10	41.35	60.12
Pool above sea-level, feet	709.65	717.83	725.83	737.26	749.47
Underlying material	Rock	Gravel	Gravel	Gravel	Gravel
Completed	1841–1848	1841–1848	1844–1883	1844–1886	1856
Available length, feet	158.0 215.3	158.0 215.0	158.0 276.9	158.0 225.2	165.5
Clear width, feet	50.0 56.0	50.0 56.0	50.0 56.0	50.0 56.0	50.0
Lift, feet	6.65	8.18	8.0	11.43	12.21
Depth on upper miter-sill	8.0	8.0	7.3	7.2	3.4
Depth on lower miter-sill	7.7	6.0	6.0	5.6	3.4
Material	All sandstone masonry.				
River walls:					
Height above lower sill, feet	24.0	23.8	23.5	25.0	27.0
Height above floor, feet	24.0	23.8	23.5	25.0	27.0
Bottom width, feet	12.0	12.0	12.0	12.0	14.0
Top width, feet	6.0	6.0	6.0	6.0	6.0
Gates:					
Material	Wood	Wood	Wood	Wood	Wood
Kind	Miter	Miter	Miter	Miter	Miter
How operated	Water and steam	Water and steam	Water	Water	Hand
How filled and emptied	(A)	(A)	(B)	(B)	(C)
DAMS.					
Type (movable or fixed)	Fixed	Fixed	Fixed	Fixed	Fixed
Underlying material	Gravel	Gravel	Gravel	Gravel	Gravel
Material of dam	Timber cribs filled with riprap	Timber cribs filled with riprap	Timber cribs filled with riprap	Timber cribs filled with riprap	Timber cribs filled with riprap
Length from lock to abutment, feet	962.5	916.0	687.5	603.0	606.0
Base width, feet	60.0	60.0	60.0	60.0	56.0
COST.					
Of old locks and dams (purchased)*	$400,000.00	$425,000.00	$440,000.00	$404,000.00	$210,000.00

* Locks and dams 1 to 7 were purchased by the United States in 1896 from the Monongahela River Navigation Co. at a total cost of $2,210,000.00. Locks 1 to 4 are double.

(A) Filled and emptied by wickets in gates.

(B) Small lock filled and emptied by wickets in gates. Large lock filled by valves in floor of head-bay, discharging under miter-sill; emptied by wickets in gates.

(C) Filled by valves in floor of head-bay, discharging under miter-sill; emptied by wickets in gates.

MONONGAHELA RIVER, PA.—*Continued.*

LOCKS.					
Location	Rice's Landing	Geneva	Dunkard Creek	Hoard's Rocks	Morgantown
Reference number	6	7	8	9	10
Distance from junction with Ohio River, miles	69.32	83.82	88.0	93.3	102.5
Height of pool above sea, feet	763.57	773.37	784.37	794.0	804.0
Underlying material	Rock	Rock	Rock	Rock	Rock
Completed	1856	1883	1882–1889	1879	Under construction 1902
Available length, feet	159.4	159.0	161.7	160.0	177.0
Clear width, feet	50.0	50.0	50.0	50.0	56.0
Lift, feet	14.10	9.80	10.0	10.53	10.0
Depth on upper miter-sill, feet	4.0	5.5	6.0	6.43	8.0
Depth on lower miter-sill, feet	4.0	5.5	6.0	7.15	7.0
Material	All sandstone masonry		Cut stone	Cut stone	Concrete
River walls:					
Height above lower sill, feet	26.0	25.0	26.0	25.5	25.0
Height above floor, feet	27.0	26.0	36	27.5	27.0
Bottom width, feet	14.0	14.0	18.0	11.5	11.6
Top width, feet	6.0	8.5	7.0	7.0	11.6
Gates:					
Material	Wood	Wood	Wood	Wood	Wood
Kind	Miter	Miter	Miter	Miter	Miter
How operated	Hand	Hand	Water	Hand	Hand
How filled and emptied	(A)	(A)	(B) (C)	(D) (E)	(F) (G)
DAMS.					
Type (movable or fixed)	Fixed	Fixed	Fixed	Fixed	Fixed
Underlying material	Gravel	Gravel	Gravel and rock	Rock	Rock
Material of dam	Timber cribs filled with riprap.	Timber cribs filled with riprap.	Timber cribs filled with riprap.	Cut stone	Concrete
Length from lock to abutment, feet	631.0	525.5	600.0	400.0	440.0
Base width, feet	56.0	56.0	5.0	28.2	23.63
COST.					
Of old locks and dams (purchased)	$170,000.00	$161,000.00			
New lock and dam complete, actual	....	....	....	$436,900.00	

(A) Filled by valves in floor of head-bay, discharging under miter-sill; emptied by wickets in gates.

(B) Filled by six openings under upper miter-sill, from cross culvert fed by a plain slide-valve in each buttress; operated by power.

(C) Emptied by culverts around quoins; plain slide-valves operated by power.

(D) Filled by two culverts through breast wall, fed each by a damper-valve in buttress.

(E) Emptied by a culvert through lower river buttress. Stoney valve used.

(F) Filled by culverts under breast wall.

(G) Emptied by wickets in gates.

MONONGAHELA RIVER, PA.—*Continued.*

LOCKS.*					
Location	Morgan's Mills	Little Falls	Little Falls	Loweville	Montana
Reference number	11	12	13	14	15
Distance from junction with Ohio River, miles	105.0	110.0	112.0	116.5	124.0
Height of pool above sea, feet	816.0	828.0	838.0	848.0	858.0
Underlying material	Rock	Rock	Rock	Rock	Rock
Completed	Under const	ruction, 1902			
Available length, feet	177.0	177.0	177.0	177.0	177.0
Clear width, feet	56.0	56.0	56.0	56.0	56.0
Lift, feet	12.0	12.0	10.0	10.0	10.0
Depth on upper miter-sill, feet	8.0	8.0	8.0	8.0	8.0
Depth on lower miter-sill, feet	7.0	7.0	7.0	7.0	7.0
Material	Concrete	Concrete	Concrete	Concrete	Concrete
River walls:					
Height above lower sill, feet	27.0	27.0	25.0	25.0	25.0
Height above floor, feet	29.0	29.0	27.0	27.0	27.0
Bottom width	12.6	12.6	11.6	11.6	11.6
Top width	12.6	12.6	11.6	11.6	11.6
Gates:					
Material	Wood	Wood	Wood	Wood	Wood
Kind	Miter	Miter	Miter	Miter	Miter
How operated	Hand	Hand	Hand	Hand	Hand
How filled and emptied	(A) (B)	(A) (B)	(A) (B).	(A) (B)	(A) (B)
DAMS.*					
Type (movable or fixed)	Fixed	Fixed	Fixed	Fixed	Fixed
Underlying material	Rock	Rock	Rock	Rock	Rock
Material of dam	Concrete	Concrete	Concrete	Concrete	Concrete
Length from lock to abutment, feet	460.0	400.0	350.0	400.0	350.0
Base width, feet	25.44	25.44	23.63	23.63	23.63

* Locks and Dams Nos. 11 to 15 are under contract but not completed (1902).

(A) Filled by culverts under breast wall.

(B) Emptied by wickets in gates.

MUSKINGUM RIVER, OHIO.

LOCK.					
Location	Marietta.	Devols	Lowell	Beverly	Luke Chute
Reference number	1	2	3	4	5
Distance from Ohio River, miles	0.25	5.75	13.85	24.71	32.97
Pool above sea-level, feet	583.17	593.64	607.12	617.54 / 624.90	628.44
Completed	1879–90	1836–40	1889–91	1836–40	1836–40
Available length, feet	366	160	160	160	160
Clear width, feet	56	36	36	36	36
Lift, feet	12.78	10.47	13.48	10.42	10.90
Depth on upper miter-sill, feet	7.55	6	6	6	6
Depth on lower miter-sill, feet	3	5.43	5.50	4.52	5.50
Built of	Stone	Stone	Stone	Stone	Stone
River walls:					
Height above lower miter-sill, feet	24.16	25.96	29	25.25	27.15
Height above upper miter-sill, feet	16.16	19.23	16.02	19.99	20.64
Bottom width, feet	13	13	13	13	13
Top width, feet	7	6.5	6	6	6
Gates:					
Operated by	Hand, Turbine	Hand	Hand	Hand	Hand
Material	Wood	Wood	Wood	Wood	Wood
Type of filling-valves	4 balanced in wall	2 cylinder in wall	1 cylinder in wall	2 cylinder in wall	2 cylinder in wall
Type of discharge-valves	5 balanced in wall	6 slide in gate	2 balanced in wall	6 slide in gate	6 slide in gate
DAM.					
Type	Slope	Step	Step	Slope	Step and slope
Built on	Gravel	Rock	Rock	Rock and gravel	Rock and gravel
Built of	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap
Length, feet	480	588	848	535 / 170	546
Width at base, feet	50	34	36	50	50

NOTE.—The original locks and dams were built by the State of Ohio, between 1836 and 1840, at a cost of about $1,300,000.00. They were transferred to the United States in 1887.

MUSKINGUM RIVER, OHIO.—*Continued.*

LOCK.					
Location	Stockport	McConnelsville	Eagleport	Tylorsville	Zanesville *
Reference number	6	7	8	9	10
Distance from Ohio River, miles	39.11	48.22	56.0	66.84	75.82
Pool above sea-level, feet	640.49	650.63	661.62	670.90	688.62
Completed	1889–91	1889–91	1836–40	1888	1836–40
Available length, feet	160	160	160	160	{ 156 } { 158 }
Clear width, feet	36	36	36	36	35.5
Lift, feet	12.05	10.14	10.99	9.28	17.72
Depth on upper miter-sill, feet	6	6	8	6	4.18
Depth on lower miter-sill, feet	5.50	5.50	5.50	5.50	5.40
Built of	Stone	Stone	Stone	Stone	Stone
River walls:					
Height above lower miter-sill, feet	29.05	27.14	27.49	26.78	30.34
Height above upper miter-sill, feet	17.50	17.50	19.0	18	15.13
Bottom width, feet	13	13	13	13	13
Top width, feet	6	6	6	6	6.75
Gates:					
Operated by	Hand	Hand	Hand	Hand	Hand
Material	Wood	Wood	Wood	Wood	Wood
Type of filling-valves	2 cylinder in wall	2 cylinder in wall	2 cylinder in wall	2 cylinder in wall	6 balanced in gate
Type of discharge-valves	2 balanced in wall	2 balanced in wall	2 balanced in wall	2 balanced in wall	6 slide in gate
DAM.					
Type	Slope	Step and slope	Slope	Step	Slope
Built on	Rock	Rock and gravel	Gravel	Rock	Soft rock
Built of	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap	Timber cribs filled with rip-rap
Length, feet	482	472	515	736	514
Width at base, feet	50	{ 36 } { 46 }	50	41	42

NOTE.—The original locks and dams were built by the State of Ohio, between 1836 and 1840, at a cost of about $1,300,000.00. They were transferred to the United States in 1887.

* Double lift.

OHIO RIVER.

LOCKS.						
Location	Davis Isl'd	Merriman's Bar	Osborn	Sewickley	Freedom	Beaver
Reference number	1	2	3	4	5	6
Distance from Pittsburg, miles	4.5	9.1	10.8	18.6	23.9	28.8
Height of pool above sea, feet	703	696.875	689.134	681.393	673.752	669.026
Underlying material	Rock and gravel	Gravel	Gravel	Gravel	Gravel	Rock and gravel
Completed	1885	Under construction,			1902	1903
Available length, feet	600	600	600	600	600	600
Clear width, feet	110	110	110	110	110	110
Lift, feet	6.125 when dam No. 2 is built	7.741	7.741	7.641	4.726	7.156 when dam No. 7 is built
Depth on upper miter-sill, feet	12.125	13.741	13.741	13.641	10.726	13.156
Depth on lower miter-sill, feet	6.59 when dam No. 2 is built	6.50	6.50	6.50	6.50	6.0
Material	Cut stone	Stone, concrete, timber	Stone, concrete, timber	Stone, concrete, timber	Stone, concrete, timber	Concrete and stone
River walls:						
Height above lower sill, feet	17.75	19.24	19.24	19.14	16.22	18.65
Height above floor, feet	19.75	20.24	20.24	20.14	17.22	19.65
Bottom width, feet	11.0	12.50	12.50	12.50	12.50	11.0
Top width, feet	11.0	12.25	12.25	12.25	12.25	12.25
Gates:						
Material	Steel	Steel	Steel	Steel	Steel	Steel
Kind	Rolling	Rolling	Rolling	Rolling	Rolling	Rolling
How operated	Steam	Electric power	Electric power	Electric power	Electric power	Electric power
How filled and emptied	Wall- and gate-valves	Wall- and gate-valves	Wall- and gate-valves	Wall- and gate-valves	Wall- and gate-valves	Wall- and gate-valves
DAMS.						
Type	Chanoine wickets	Movable	Movable	Movable	Movable	Wickets and bear-trap
Underlying material	Gravel	Gravel	Gravel	Gravel	Gravel	Gravel
Material of dam	Concrete, stone, and timber	Concrete, stone, and timber	Concrete, stone, and timber	Concrete, stone, and timber	Concrete, stone, and timber	Concrete, stone, and timber
Length from lock to abutment, feet	1223	1000, appr.	1000, appr.	1000, appr.	1000, appr.	1000
Weirs:						
Dimensions	(*)					3 of 120 ft. each
Closure	Wickets and bear-trap					1 A-frame 2 bear-traps
Level of sill to lower pool, feet	Bear-trap, 3.2 Weir No. 1, 4 " " 2, 3					6
Worked from bridge or boat	No. 1 boat No. 2 bridge					Hand and waterpower
Navigation pass:						
Dimensions, feet	12 deep 41.5 wide 719 long 12.125 high					14 deep 80 wide 600 long 13.156 high
Closure	Chanoine wickets					Chanoine wickets
Level of sill to lower pool, feet	6					6
Worked from	Boat					Boat
COST.						
New lock and dam complete, estimate		$750,000	$750,000	$750,000	$750,000	$900,000
New lock and dam complete	about $1,000,000					

Operating Force.—At Lock No. 1 eight men are employed, at wages from $35 to $75 per month.

(*) Bear-trap: 52′ long, 41.5′ wide, 9.33′ high.
Weir No. 1: 212′ long, 41.5′ wide, 10′ high.
Weir No. 2: 215′ long, 41.5′ wide, 9′ high.

ST. MARY'S FALLS CANAL, MICH.

(Between Lakes Superior and Huron.)

Lock.	Old (Weitzel) Lock.	New (Poe) Lock.
Built on	Rock	Rock
Built of	Cut stone	Limestone
Completed	1881	1896
Extreme length, feet	717	1124
Length between quoins, feet	515	800
Clear width, feet	80	100
Lift, feet	18, ordinary	18, ordinary
Depth on upper miter-sill, feet	16	21
Depth on lower miter-sill, feet	16	21
Wall height above floor, feet	....	43.5
Material of floor	....	Plank
Type of gates	Miter, bowstring trussed	Miter, full arched
Material of gates	Wood	Steel
Operating power	Hydraulic	Hydraulic
Location of filling-valves	....	In upper miter wall
Location of emptying-valves	....	In lower miter wall
Type of filling-valves	....	Vertical balanced
Type of emptying-valves	....	Vertical balanced
Total original cost of improvements	$2,000,000.00	$3,700,000.00

COLBERT SHOALS CANAL, ALA. (TENNESSEE RIVER).

Colbert Shoals Canal	Lift lock.
Location	29 miles west of Florence, Ala., at Riverton
Pool above sea-level, feet	383.370
Built on	Rock
Built of	Cut masonry, chiefly limestone
Completed	Masonry, 1897
Entire length, feet	551.0
Available length, feet	340.0
Clear width, feet	80.0
Lift, feet	26.0 max.
Depth on upper miter-sill, feet	7.0
Depth on lower miter-sill, feet	7.0
Height of wall above lower miter-sill, feet	36.25
Height of wall above floor, feet	37.0
Material of floor	Bed-rock
Width of wall at floor, feet	21.5
Width of wall at coping, linear feet	5.0
Type of gates	Miter
Material of gates	Steel
Operated by	Machinery
Location of filling-valves	In walls
Location of emptying-valves	In walls
Type of filling-valves	Balanced butterfly (steel)
Type of emptying-valves	Balanced butterfly (steel)
Number and size of filling-valves	Two each 8 ft. × 10 ft.
Number and size of emptying-valves	Two each 8 ft. × 10 ft.
Type of head-bay coffer	None
Type of tail-bay coffer	Steel gates

NOTE.—Canal not yet available for commerce, being in course of construction (1902).

MUSCLE SHOALS CANAL (TENNESSEE RIVER).

LOCK.	Designation.					
	"A"	"B"	1	2	3	4
Location	Elk River Shoals	Elk River Shoals	Big Muscle Shoals	Big Muscle Shoals	Big Muscle Shoals	Big Muscle Shoals
Number	A	B	1	2	3	4
Miles from Florence, Ala.	29.66	28.16	20.16	18.21	16.31	14.50
Pool above sea-level, feet	{ 534.061 / 526.261	521.945	505.665 / 515.665 }	505.665	499.325	487.820
Built on	Rock	Rock	Rock	Rock	Rock	Rock
Built of	Limestone	Limestone	Limestone	Limestone	Limestone	Limestone
Completed	1888	1888	1889	1889	1889	1889
Extreme length, feet	407	407	411	409	453	445
Available length, feet	285	285	285	285	285	285
Clear width, feet	60	60	60	60	60	60
Lift, feet	4.32 to 12.2	13 max.	10 max.	6.34	11.50	10.61
Depth on upper miter sill, feet	5.4 to 13.2	6.0	5.0 min.	7.25	7.00	6.5
Depth on lower miter-sill, feet	5.00	2.5 to 19.3	5.0	5.00	5.00	5.0
Wall height above lower miter-sill, ft.	17.60	19.25	16.65	13.09	18.50	17.10
Wall height above floor, feet	19.75	20.12	18.62	16.00	20.30	19.75
Material of floor	Rock	Rock	Rock	Rock	Rock	Rock
Chamber-wall width at floor line, feet.	11.0	11.0	10.5	7.5	10.5	10.5
Chamber-wall width at coping, feet	4.0	4.0	4.0	4.0	5.0	5.0
Type of gates	Mitering	Mitering	Mitering	Mitering	Mitering	Mitering
Material of gates	Iron	Iron	Iron	Iron	Iron	Iron
Operating power	Hydraulic	Hand	Hand	Hand	Hand	Hand
Location of filling-valves	In wall	In wall	In wall	In wall	In wall	In wall
Location of emptying-valves	In wall	In wall	In wall	In wall	In wall	In wall
Size of filling-valves, feet	4½×6	4½×6	4½×6	4×6	3¾×7	3¾×7
Size of emptying-valves, feet	4½×6	4½×6	4½×6	4×6	3¾×7	3¾×7
Type of filling-valves	Vertical lift	Balance	Balance	Balance	Balance	Balance
Type of emptying-valves	Vertical lift	Balance	Balance	Balance	Balance	Balance
Number of men employed	1	1	1	1	1	1
Wages per month	$35	$30	$40	$35	$35	$35
Original cost	$104,907	$137,694	$116,573	$66,055	$137,694	$115,287

MUSCLE SHOALS CANAL (TENNESSEE RIVER).—*Continued.*

LOCK.	Designation.				
	5	6	7	8	9
Location	Big Muscle Shoals	Big Muscle Shoals	Big Muscle Shoals	Big Muscle Shoals	Big Muscle Shoals
Number	5	6	7	8	9
Miles from Florence, Ala.	12.64	8.48	7.13	6.37	6.00
Pool above sea-level, feet	477.208	465.263	452.283	440.325	430.248
Built on	Rock	Rock	Rock	Rock	Rock
Built of	Limestone	Limestone	Limestone	Limestone	Limestone
Completed	1889	1889	1890	1890	1889
Extreme length, feet	378	378	378	378	378
Available length, feet	285	285	285	285	285
Clear width, feet	60	60	60	60	60
Lift, feet	11.94	12.98	11.95	10.07	9.75
Depth on upper miter-sill, feet	7.0	7.0	7.0	6.9	7.0
Depth on lower miter-sill, feet	5.0	5.0	5.0	5.0	5.0 min.
Wall height above lower miter-sill, ft.	18.94	19.98	18.96	17.08	17.50
Wall height above floor, feet	21.25	23.75	20.75	19.00	19.50
Material of floor	Rock	Rock	Rock	Rock	Rock
Chamber-wall width at floor line, feet.	11.0	11.0	11.0	10.5	10.5
Chamber-wall width at coping, feet	4.0	4.0	4.0	4.0	4.0
Type of gates	Mitering and drop	Mitering and drop	Mitering and drop	Mitering and drop	Mitering and drop
Material of gates	Iron and wood	Iron and wood	Iron and wood	Iron and wood	Iron and wood
Operating power	Hand	Hand	Hand	Hand	Hand
Location of filling-valves	In wall	In wall	In wall	In wall	In wall
Location of emptying-valves	In wall	In wall	In wall	In wall	In wall
Size of filling-valves	4½×6	4½×6	4½×6	4½×6	4½×6
Size of emptying-valves	4½×6	4½×6	4½×6	4½×6	4½×6
Type of filling-valves	Balance	Balance	Balance	Balance	Balance
Type of emptying-valves	Balance	Balance	Balance	Balance	Balance
Number of men employed	1	1	1	1	1
Wages per month	$35	$35	$35	$35	$35
Original cost	$137,407	$148,611	$137,407	$115,000	$115,000

WABASH RIVER, IND., AND ILL.

LOCK.

Location	Mt. Carmel, Ill.
Number	(Only one lock on the river)
Miles from the Ohio River	92.9
Built on	Rock
Built of	Masonry
Completed	1894
Extreme length, feet	314.6
Available length, feet	214.0
Clear width, feet	52.0
Lift, feet	11.5
Depth on upper miter-sill, feet	5.08
Depth on lower miter-sill, feet	3.5
Height of wall above lower miter-sill, feet	25.0
Height of wall above floor, feet	27.0
Material of floor	Rock
Width of wall at floor, feet	13 and 16
Width of wall at coping, feet	7 and 16
Type of gates	Miter
Material of gates	Wood
Operating power	Hand
Location of filling-valves	Wall
Location of emptying-valves	Wall
Total gross area of filling-valves (2 valves), square feet	49.2
Total gross area of emptying-valves (2 valves), square feet	49.2
Type of filling-valves	Vertical balanced
Type of emptying-valves	Vertical balanced
Type of head-bay coffer	Needle dam
Type of tail-bay coffer	None
Number of men employed	Two
Wages per month	$45 and $50
Original cost, lock and dam	$260,000.00, about

DAM.

Type	Fixed
Step or slope	Slope
Built on	Rock and gravel
Built of	Timber cribs filled with rip-rap
Extreme length, feet	1100
Extreme width of base, feet	50

APPENDIX B.

SPECIFICATIONS.

In the following pages will be found examples of specifications relating to the construction of works pertaining to the improvement of rivers.* They have been made comprehensive, so as to include as far as practicable those points about which experience has shown that disputes are liable to arise, as well as those requiring a special description of method.

Certain clauses are usually included in all Government formal contracts; these are given under the heading of "General Instructions for Bidders," and "General Conditions," preceded by the "Advertisement."

ADVERTISEMENT.

U. S. Engineer Office,.................................., 19...

Sealed proposals for building..will be received at this office until..........., standard time,...........19..., and then publicly opened. Information will be furnished on application to......................., Corps of Engineers, U. S. A.

SPECIFICATIONS.

GENERAL INSTRUCTIONS FOR BIDDERS.

1. The attention of bidders is especially invited to the Acts of Congress approved February 26, 1885, and February 23, 1887, as printed in vol. 23, page 332, and vol. 24, page 414, United States Statutes at Large, which prohibit the importation of foreigners and aliens, under contract or agreement, to perform labor in the United States or Territories of the District of Columbia.

2. Preference will be given to articles or materials of domestic production, conditions of quality and price being equal, including in the price of foreign articles the duty thereon.

* These specifications have been compiled from recent contracts made under direction of the Corps of Engineers, U. S. Army, and are reproduced by permission of the Chief of Engineers.

3. No proposal will be considered unless accompanied by a guaranty which should be in manner and form as directed in these intructions.

4. All bids and guaranties must be made in triplicate, upon printed forms to be obtained at this office.

5. The guaranty attached to each copy of the bid must be signed by an authorized surety company, or by two responsible guarantors, to be certified as good and sufficient guarantors by a Judge or clerk of a United States Court, United States District Attorney, United States Commissioner, or Judge or clerk of a State court of record, with the seal of said court attached.

6. A firm as such will not be accepted as surety, nor a partner for a copartner or firm of which he is a member. Stockholders who are not officers of a corporation may be accepted as sureties for such corporation. Sureties, if individuals, must be citizens of the United States.

7. When the principal, a guarantor, or a surety is an individual, his signature to a guaranty or bond shall have affixed to it an adhesive seal. Corporate seals will be affixed by corporations, whether principals or sureties. All signatures to proposals, guaranties, contracts, and bonds should be written out in full, and each signature to guaranties, contracts, and bonds should be attested by at least one witness, and, when practicable, by a separate witness to each signature.

8. Each guarantor will justify in a sum of................dollars ($.......). The liability of the guarantors and bidder is determined by the Act of March 3, 1883, 22 Statutes, 487, Chap. 120, and is expressed in the guaranty attached to the bid.

9. A proposal by a person who affixes to his signature the word "president," "secretary," "agent," or other designation, without disclosing his principal, is the proposal of the individual. That by a corporation should be signed with the name of the corporation, followed by the signature of the president, secretary, or other person authorized to bind it in the matter, who should file evidence of his authority to do so. That by a firm should be signed with the firm name, either by a member thereof or by its agent, giving the names of all members of the firm. Any one signing the proposal as the agent of another or others must file with it legal evidence of his authority to do so.

10. The place of residence of every bidder, and post-office address, with county and State, must be given after his signature.

11. All prices must be written as well as expressed in figures.

12. One copy each of the advertisement, the instructions for bidders, and the specifications, all of which can be obtained at this office on application by mail or in person, must be securely attached to each copy of the proposal and be considered as comprising a part of it.

13. Proposals must be prepared without assistance from any person employed in or belonging to the military service of the United States or employed under this office.

14. No bidder will be informed, directly or indirectly, of the name of any person

intending to bid or not to bid, or to whom information in respect to proposals may have been given.

15. All blank spaces in the proposal and bond must be filled in, and no change shall be made in the phraseology of the proposal, or addition to the items mentioned therein. Any conditions, limitations, or provisos attached to proposals will be liable to render them informal and cause their rejection.

16. Alterations by erasure or interlineation must be explained or noted in the proposal over the signature of the bidder.

17. If a bidder wishes to withdraw his proposal, he may do so before the time fixed for the opening, without prejudice to himself, by communicating his purpose in writing to the officer who holds it, and, when reached, it shall be handed to him, or his authorized agent, unread.

18. No bids received after the time set for opening of proposals will be considered.

19. The proposals and guaranties must be placed in a sealed envelope marked "Proposals for.., to be opened.........19..," and inclosed in another sealed envelope addressed to......................, Corps of Engineers, U. S. A.,.., but otherwise unmarked. It is suggested that the inner envelope be sealed with sealing wax.

20. It is understood and agreed that the quantities given are approximate only, and that no claim shall be made against the United States on account of any excess or deficiency, absolute or relative, in the same. Bidders, or their authorized agents, are expected to examine the maps and drawings in this office, which are open to their inspection, to visit the locality of the work, and to make their own estimates of the facilities and difficulties attending the execution of the proposed contract, including local conditions, uncertainty of weather, and all other contingencies.

21. The United States reserves the right to reject any and all bids, and to waive any informality in the bids received; also to disregard the bid of any failing bidder or contractor known as such to the Engineer Department, or any bid which is palpably unbalanced or obviously below what the work can be done for, or the bid of any bidder who shall fail to produce, when called upon to do so, evidence satisfactory to the engineer officer in charge of the said bidder's ability to do the contemplated work within the required time, including his control of the necessary means and equipment. The failure of a bidder to make satisfactory progress or to complete on time similar work under previous contracts with the United States will be duly considered in canvassing bids, and may be a valid cause for the rejection of his proposal. Reasonable grounds for supposing that any bidder is interested in more than one bid for the same item will cause the rejection of all bids in which he is interested.

22. The bidder to whom award is made will be required to enter into written contract with the United States, with good and approved security, in an amount of twenty (20) per cent of the amount of his bid, within ten (10) days after being notified of the acceptance of his proposal. The contract which the bidder and guarantors

promise to enter into shall be, in its general provisions, in the form adopted and in general use by the Engineer Department of the Army, blank forms of which can be inspected at this office, and will be furnished, if desired, to parties proposing to put in bids. Parties making bids are to be understood as accepting the terms and conditions contained in such form of contract.

23. The sureties, if individuals, are to make and subscribe affidavits of justification on the back of the bond to the contract, and they must justify in amounts which shall aggregate double the amount of the penal sum named in the bond.

24. Bidders are invited to be present at the opening of the bids.

GENERAL CONDITIONS.

25. A copy of the advertisement, and of the specifications, instructions, and conditions will be attached to the contract and form a part of it.

26. The contractor should, within ten days from the award of the contract, furnish the Engineer office with the post-office address to which communications should be sent.

27. Transfers of contracts, or of interest in contracts, are prohibited by law.

28. The contractor will not be allowed to take advantage of any error or omission in these specifications, as full instructions will always be given should such error or omission be discovered.

29. The decision of the Engineer Officer in charge as to quality and quantity shall be final.

30. Payments will be made on monthly estimates for the work done during the month, unless otherwise herein specified. A percentage of ten (10) per centum will be reserved from each payment until the completion of the contract.

31. Unless extraordinary and unforeseeable conditions supervene, the time allowed in these specifications for the completion of the contract to be entered into is considered sufficient for such completion by a contractor having the necessary plant, capital, and experience. If the work is not completed within the period stipulated in the contract, the Engineer Officer in charge may, with the prior sanction of the Chief of Engineers, waive the time limit, and permit the contractor to finish the work within a reasonable period, to be determined by the said Engineer Officer in charge. Should the original time limit be thus waived, all expenses for inspection and superintendence and other actual loss and damages to the United States due to the delay beyond the time originally set for completion shall be determined by the said Engineer Officer in charge and deducted from any payments due or to become due to the contractor. *Provided*, *however*, that the party of the first part may, with the prior sanction of the Chief of Engineers, waive for a reasonable period the time limit originally set for completion and remit the charges for expenses of superintendence and inspection for so much time as in the judgment of the said Engineer Officer in charge may actually have been lost on account of unusual freshets, ice, rainfall, or other abnormal force or violence of the elements, or by epidemics, local or State quarantine restrictions, or other

unforeseeable cause of delay arising through no fault of the contractor, and which prevented him from commencing or completing the work or delivering the materials within the period required by the contract: *Provided, further*, that nothing in these specifications shall affect the power of the party of the first part to annul the contract as provided in the form of contract adopted and in use by the Engineer Department of the Army.

32. The contractor will be required to hold the United States harmless against all claims for the use of any patented article, process, or appliance in connection with the contract herein contemplated.

SPECIFICATIONS FOR A LOCK-TENDER'S DWELLING (FRAME) AND FENCING.

SPECIAL DESCRIPTION.

1. **Description of Work.**—The contract to be entered into is for furnishing all labor, materials, tools, appliances, and work of every kind required to build and complete ready for occupation one frame dwelling-house, outhouse, and cistern; and to supply and erect a fence around the United States property (or a portion of the United States property) at the site of Lock No....., ...River, and in accordance with the drawings exhibited and these specifications.

2. **Location.**—Lock No..... is located on the...........................River, near Station, in...........County, State of......................, and on the ....Railway. The lock site is about......miles from the mouth of the river, and is accessible by water except in the low-water season.

The exact positions and elevations of all work will be given by the Engineer in charge of the work for the United States, and all work must be built at the places and in the manner designated in these specifications, in the drawings, and by further explanatory information, if such is needed.

3. **Drawings.**—Drawings showing the method of construction are on file, and are hereby made a part of these specifications. No deviation from them shall be made unless specially permitted in writing by the Engineer.

(1) DWELLING AND OUTHOUSE.

4. **Excavation.**—Excavation will include the material to be removed for the cellar, cistern, drains, outhouse, vault and foundations of the walls, chimneys, piers, etc. It must conform to such lines, depths, and slopes as may be given by the Engineer.

The excavated material shall be used, so far as required, to grade the lot about the buildings, the surplus, if any, being deposited where directed by the Engineer, but at distances not exceeding .... feet from the house.

5. **Foundations.**—The foundation and cellar walls, chimney bases, and piers shall be built of brick, upon footing courses of concrete 12 inches deep, extending in every direction 4 inches beyond the base of the brick-work.

The top of the concrete footings for the foundation walls, chimney bases, and piers shall in no case be less than......inches below the finished grade line of the house lot, and the cellar walls shall extend......feet above the tops of their concrete footings.

The outhouse sills shall rest on brick piers, as hereinafter specified. Suitable openings with standard cast-iron gratings shall be provided in foundation walls and for windows in cellar wall.

6. **Concrete.**—All cement shall be Portland. The footing courses under walls, chimneys, and piers, together with the cellar floor and the porch steps, shall be composed of cement concrete, mixed in the proportion of one barrel of cement, two barrels of clean sand, and four barrels of clean screened broken stone, of size to pass through a 1½-inch ring. The cement, sand, and broken stone must be satisfactory to the Engineer.

The stone shall be spread upon a clean board platform, and thoroughly moistened. The sand and cement must be thoroughly mixed while dry, and sufficient clean water added to bring the mortar to the proper consistency. The mortar must be immediately added to the stone, and the mass turned with shovels until properly mixed. The concrete must be deposited and well rammed before the initial set of the cement takes place.

The cellar floor shall be given a slope for drainage as directed. It and the porch steps shall be covered with a layer of cement mortar, 1 inch thick, composed of 1 part of cement to 2 parts of sand, floated on before the concrete has set. The utmost cleanliness will be required in mixing and placing the concrete and mortar.

7. **Brick-work, Cistern, and Chimneys.**—The foundation and cellar walls, piers, vault, cistern, and chimneys shall be built in a workmanlike manner, of hard-burned brick laid in cement mortar, as shown on the plans, and as directed. The cement must be fresh and of quality similar to that used for the concrete, and the sand must be clean and sharp, and both satisfactory to the Engineer. The mortar shall be mixed in proportion of one of cement to two of sand by volume.* The bricks must be moistened when laid. The exposed faces of walls, piers, and chimneys shall be of selected, smooth, hard-burned brick, neatly pointed, and the tops of the chimneys shall be finished out as shown and be well flashed with best grade tin, painted two coats on both sides. Flues shall be neatly pointed throughout, with a small trowel. A cast-iron thimble, 7 inches in diameter, with tin cover, shall be set in the kitchen flue. Proper wrought-iron arch bars shall be built in where required.

The cistern shall be built of brick, one layer thick, laid flat and in a circle, the earth

---

* If lime mortar is to be used, the proportions should be 1 of lime to 2½ or 3 of sand, and the lime should be specified as "fresh and of the best quality." It is preferable, however, to use cement throughout, and Portland is to be preferred to natural, if the small extra expense is not an objection.

being wetted and tamped well behind as the wall is built up. The bottom shall be an invert, also of brick. Toward the top the brick shall be carefully arched and finished with 20″ cast-iron ring and cover of approved pattern. A brick filter having inside dimensions of 8″×8″ shall be built from the floor to a point 2 feet below the ring. The entire inner surface of the cistern, including the bottom, shall be well and neatly plastered with two coats of cement mortar, mixed one part of cement and two parts of sand. The contractor shall furnish and set above the cistern, with all its fittings and pipe, as required by the Engineer, a pump equal in all respects to.............., with brass cylinder, and connect the cistern with the downspouts of the house by six-inch vitrified tile.

8. Fireplaces.—There will be.........fireplaces in the house, as shown on plans; ....of these shall have hearths made of concrete, covered with a ¾″ layer of cement mortar as directed by the Engineer, 21 inches wide and 54 inches long, and shall have their facing of plastering; the other two shall have facings and hearths of enameled tile. The hearths will be 21″×54″ set in a proper and workmanlike manner. All fireplaces shall be furnished with 20-inch grates, with cast enameled fronts, ash pans, and fenders of approved design. The grates shall be well set in fire brick with fire clay for mortar, and the back walls shall be sloped or made concave as directed.

9. Mantels.—The mantels shall be quartered oak. Three of them shall have 54″ shelf and be 47″ high, and be without tops and equal in all respects to Nos............ of.......catalogue. Two of them shall have tops and shall be in all respect equal to Nos..........of same catalogue, with French bevel mirror, and accompanying fittings as therein shown.

10. Framing.—All framing stuff is to be of sound, well-sawed pine or poplar, of sizes shown generally by drawings exhibited, fitted and put together in a strong, workmanlike manner.

11. Sills, etc.—Sills, corner posts, and bracing shall be framed and connected as shown or as directed by the Engineer. The sills shall be set on the walls in mortar.

12. Joists.—Joists shall be doubled and framed at chimney, stairs, and other openings, and under partitions, all properly brought to level and well spiked. Cross bridging shall be used in rows spaced not more than 6 feet apart. Spaces between joists over sills shall be closed with 2-inch plank and made mouse-tight. Joists shall be generally 16 inches between centers.

13. Studding, etc.—Studding shall be generally 2″ × 4″, and set 16″ between centers, and shall be doubled on the sides of all openings, and shall extend in one length from plate to sill. It shall be trebled at partition connections. Corners shall be of single 6″ × 6″ stuff, in one length, properly braced to sills and plates. Plates shall be of 2″ × 4″ stuff, doubled, well spiked, and breaking joints.

14. Weather-boarding.—The houses shall be covered with first-class, well-seasoned yellow poplar or yellow pine weather-boarding, 5¾ inches wide, 4½ inches to weather,

properly nailed to each stud, and laid on sound single-dressed $\frac{7}{8}''$ sheathing; the latter is to be covered with 3-ply roofing paper, of best quality.

15. **Cornices.**—Cornices shall be as shown on drawings; the lumber shall be first-class, of same quality and material as weather-boarding, dressed on both sides, neatly got out and well put up.

16. **Roofs.**—Rafters shall be notched on and well nailed to plates and collar beams, and set generally 16″ between centers. Roof sheathing shall be sound poplar or pine, close laid, well nailed, and breaking joints. The roof of the main house and porches shall be covered with No. 1 cypress shingles, laid $4\frac{1}{2}$ inches to the weather.

17. **Gables.**—The gables on the main house shall be finished as shown, and shall be covered with No. 1 cypress dimension shingles, laid on sheathing. Each gable shall have an ornament, as shown, similar to No......on page.....of.............catalogue.

18. **Gutters and Down Spouts.**—No. 27 galvanized iron gutters, supported by wrought-iron brackets, etc., as shown, shall be attached to the eaves of the main house and back porch. The gutter on the front porch shall be built in the cornices as shown, and lined with the best quality tin, laid with flat seams, painted two coats on both sides, and thoroughly flashed to make a water- and snow-tight job. All gutters shall be connected with 4-inch corrugated No. 27 galvanized iron down spouts, as shown or as directed. A slope of $\frac{1}{16}$ inch to 1 foot shall be given to all gutters. Down spouts must have proper cut-offs, elbows, and fastenings, and connect with the tile drains leading to the cistern and elsewhere as directed.

19. **Valleys and Flashings.**—Valleys, chimneys, outside doors, and windows, (where not protected by porch roofs,) porch gutter, and porch roof, shall be well flashed with best grade tin, painted two coats on both sides. Flashing must make a thoroughly water- and snow-tight job.

20. **Floors.**—Floors on the first and second stories shall be of first-class, seasoned, long-leaf yellow pine, $\frac{13}{16}'' \times 5\frac{1}{4}''$ flooring. The porch floors shall be of selected stuff, free from sap and pitch, and laid in white lead. All floors shall be properly laid and blind nailed. The attic shall be floored with No. 1 common grade $\frac{13}{16}'' \times 5\frac{1}{4}''$ yellow-pine flooring. No flooring shall be laid until the windows are in place, and the house has been made water-tight.

21. **Doors and Transoms.**—The doors, except the front door, shall be ogee four panel, white or yellow pine, or cypress of dimensions shown on drawings, and shall be good, selected "second quality" doors, all $1\frac{3}{8}$ inches thick. The front door shall be a $3' \times 7' \times 1\frac{3}{8}''$ first quality sash door, similar to No.....on page.....of.................. catalogue. It shall be glazed with double thickness glass, plain. Glazed transoms, properly hung, with necessary fixtures, shall be placed where shown on the drawings. The door frames shall be of pine or poplar, well made, and set in a workmanlike manner.

22. **Windows.**—Windows, except in the cellar, shall have pine or poplar box frames, properly made and set. The sash shall be check rail $1\frac{3}{8}$ inch, first quality white or yellow

pine, in sizes shown on the plans, glazed with double-strength glass, double hung with cast-iron weights and phosphor-bronze chain.

The cellar windows shall have white oak frames $1\frac{3}{4}''$ thick, and 3-light $10'' \times 12''$ glazed sash $1\frac{3}{8}''$ thick as shown. These windows must be properly hung on hinges, with buttons, hooks, etc., complete.

23. **Stairs.**—The main stairway shall be of oak, constructed as shown on plans and detailed drawings. Cellar stairs shall be of oak, with open steps, and dressed hand rail. Attic stairs shall be of hard pine or poplar.

Stairs and steps shall be built with three horses under each set. All stairs must be stiff, solid, and properly constructed.

24. **Porches.**—There shall be two porches on the main house, built as shown on plans and detailed drawings, and floored and roofed as herein specified.

25. **Ceiling.**—The ceiling lumber shall be first-class, dressed, beaded, tongue-and-grooved long-leaf yellow pine, $\frac{5}{8}'' \times 3\frac{1}{4}''$ wide. The walls of the kitchen shall be ceiled, except the chimney, and the porches shall be ceiled overhead. Ceiling must be well and closely set, and blind nailed.

26. **Shelves and Hook Strips.**—Neat, well fastened, dressed shelves shall be put up in the cellar, kitchen, pantry, and all closets as directed. About 150 linear feet of $\frac{7}{8}'' \times 12''$ lumber will be needed for these shelves. Dressed strips $\frac{7}{8}'' \times 3''$, with bronzed wire clothes hooks, spaced 6″ apart, shall be fitted in all closets, as directed; about......feet will be needed for each closet.

27. **Inside Finish.**—Inside finish shall be of yellow pine, oak, or poplar, except the main stairway, and shall be put on after the finishing coat of plastering. Bases shall be $\frac{7}{8}$ inch $\times 7\frac{1}{2}$ inches; base shoes, $\frac{3}{8}$ inch $\times 1\frac{1}{8}$ inch; casing, $\frac{7}{8}$ inch $\times 5$ inches; and window stools and aprons, $\frac{7}{8}$ inch $\times 4\frac{1}{2}$ inches. Door and window casings shall have the proper corner and base blocks, and the partition rails in transom doors shall match the casing. All shall be of pattern to be approved by the Engineer. All finishing lumber, not otherwise specified, is to be supplied and placed as shown or directed.

28. **Outside Blinds.**—Outside blinds of suitable size shall be furnished for all windows except those in the cellar and attic, and shall be of white pine $1\frac{3}{8}$ inches thick, rolling slats, and fitted with sill-catches and hold-back hinges similar to......, of...... catalogue.

29. **Fly-screens.**—Screen doors and windows, of pattern to be approved by the Engineer, shall be provided and set up on all doors and windows except........, of standard mesh galvanized wire netting, in white or yellow pine frames, well painted. Doors to be provided with hooks and eyes and hold-back spring hinges, and latches similar to ...., of.........catalogue.

30. **Lath and Plastering.**—The entire house, except cellar, kitchen, and attic, shall be lathed and plastered, and the chimney in the kitchen shall be plastered. Lath must be of first-quality sound pine or poplar, well put on, and the plastering shall be first-quality 3-coat work. The first and second coats shall be composed of one measure of

good, fresh, white lime, to four of clean screened sand, and one-third of a measure of hair. The last or skin coat must contain no sand, but enough plaster of Paris to make it clear white, and to finish with an even gloss. Each coat must be dry before another is put on, and the first coat must be well scratched. All the lime for plastering shall be passed through a suitable screen after being slaked. The mortar for the first and second coats shall be mixed at least eight days before it is put on. Plastering shall be retouched if necessary after the inside finishing is done.

31. **Hardware.**—The numbers given refer to the catalogue of................and all hardware must be of the same numbers or their equal in all respects. All locks, except for the front door, shall be mortise, and shall be provided with keys, knobs, escutcheons, etc., and shall be of antique copper finish, similar to No........ The lock for the front door shall be similar to No......, and be provided with all fittings. The closet doors shall be hung on $2\frac{1}{2}'' \times 2\frac{1}{2}''$, and the other doors on $4'' \times 4''$ loose-pin butts No....... Transoms shall be hung on No.......butts, $\frac{1}{4}'' \times 2'' \times 2''$, and be fitted with No.........lifters $\frac{5}{16}'' \times 4'$. All windows shall be fitted with sash fasteners No......., and sash lifts No........

There must be furnished and set up in a workmanlike manner, where directed in the kitchen, one cast-iron enameled sink $16'' \times 28'' \times 6''$, with back, similar to No....., connected with the drain by a lead full-S trap and $1\frac{1}{4}''$ galvanized iron pipe; a brass force-pump of $2\frac{1}{2}$-inch cylinder, equal to No....., must also be furnished and set up at the sink, and connected with the cistern by an underground galvanized iron pipe as required.

32. **Outbuilding.**—There shall be an outbuilding arranged and built in accordance with plans shown, and placed where directed by the Engineer. It shall be built in workmanlike manner throughout, and shall rest on brick piers extending......feet below the finished surface of ground, and shall have a circular brick-lined vault as shown. It shall be built of the same general class of lumber as the main house and be ceiled with second grade $\frac{5}{8}'' \times 3\frac{1}{4}''$ beaded ceiling, and floored with $1\frac{1}{8}'' \times 3\frac{1}{4}''$ second grade matched yellow-pine flooring, except the coal bin, which shall be ceiled and floored with $1\frac{1}{2}''$ undressed oak not matched. There shall be......window with fixed slats,......with a hinged shutter, and......with double check-rail, white or yellow pine sash as shown. The latter shall be hung and fastened similarly to the dwelling-house windows. The doors shall be as shown, and similar in quality to those in the interior of the main house. Door and window frames shall be properly made and set. The doors shall have rim locks with porcelain knobs, similar to No....., with loose-pin butts $4'' \times 4''$, No...... The gutters and down spouts shall be similar in quality and workmanship to those used on the dwelling, and shall be properly hung and fastened, and be connected with the outside drains. This outhouse shall be weather-boarded, shingled, trimmed, and painted to match the dwelling-house.

33. **Painting.**—All paints shall be mixed at the site from pure white lead, equal to the "Anchor" brand, and pure linseed oil, with first-quality ground-in-oil colors to be selected by the Engineer. The outside of the house, outbuilding, and yard fence shall

be painted three coats. All the outside finished work must be primed as it is put up; the priming to count as one coat. The inside of the outbuilding and the inside of the kitchen and pantry shall be painted two coats. All other inside wood-work shall be filled, stained as directed, and hard-oiled with best grade of material on surface rubbed smooth. All nail-holes shall be well puttied.

34. **Drains, Walks, etc.**—Vitrified tile drains shall be laid as shown on the drawings, and of the sizes thereon given. The joints shall be carefully placed, cemented, and scraped off inside, and the earth shall be well tamped around the tiling, all of which work must be done to the satisfaction of the Engineer.

Cement walks 3 feet wide, without curbs, shall be laid where shown on the drawings. For these walks trenches shall be excavated 12 inches deep below the finished grade, and the bottom of these shall be well rammed and then covered with fine broken stone, gravel, or cinders, also well rammed, up to 5 inches below the finished grade. The bed thus made shall be covered with 4 inches of concrete of the same composition as that in the house foundations, well rammed and surfaced with 1 inch of mortar composed of 1 part of cement to 2 parts of sand, all brought up to the required grade, rounded off, and divided into blocks 3 feet long cut through the concrete. This sidewalk must be laid and finished in the most approved manner by a skilled sidewalk mason.

FENCE.

35. **Materials.**— There will be two kinds of fence used; that for the property lines and roads shall be of post and wire construction, and that for the house grounds of posts and pickets, all as shown on the drawings. Of the former there will be about .....rods, and of the latter about.....rods. The wire must be of the style known as ...., shown on page......, catalogue of...................., for 190 , or in every particular equal to it. All posts shall be of red cedar or locust, straight, sound, and free from imperfections injurious to durability. Those for the wire fence shall be round, 7 feet 6 inches long, and 6 inches in least diameter at the top. Those for the picket fence shall be 6 feet 6 inches long, sawed tapering, $4'' \times 6''$ at the bottom, and $2'' \times 6''$ at the top. The pickets and rails shall be of No. 1 yellow poplar or yellow pine, dressed throughout. The braces for the wire fence shall be of undressed white oak, yellow poplar, or yellow pine.

36. **Construction.** — The posts for the wire fence shall be placed one rod apart, center to center, and shall stand above ground 4 feet 6 inches. Those for the picket fence shall be spaced 8 feet apart, center to center, and show above ground 3 feet 6 inches, the portion showing being dressed. All posts shall be well tamped and properly lined, horizontally and vertically, and their tops shall be sawed off with a slope, as shown. The rails shall be securely spiked to the posts with 20^d wire spikes and the pickets to the rails with 8^d wire nails. The wire shall be securely fastened to the posts with staples, after being tightly stretched with suitable appliances. All angles shall

be properly braced and tied, and......gates of the style shown on the drawings, with hinges and approved latches, shall be placed where directed.

The picket fence with its gates shall be primed as its construction progresses, and when completed shall receive two coats of paint, of quality and color similar to that used on the main portion of the dwelling.

The price bid per lineal rod for the fencing must include the cost of painting.

37. **Complete Job Required.**—It is the object and intention of the contract to embrace the entire completion of the house, outbuilding, and fencing in a suitable workmanlike manner, ready for use, and the prices named in the proposal must be based accordingly; the contractor will be required thus to complete the work regardless of any omissions as to minor details in the drawings or specifications.

38. **Contractor to Assume Risks.**—It is understood that the contractor is to assume all risks to the buildings and material from fire or other causes during construction; and it is hereby agreed that in case of accident or damage to the buildings or material, by fire or otherwise, before the final completion and acceptance of the work embraced by the contract, the contractor shall, at his own expense, restore the work to its original condition before the damage or accident occurred. The United States will not be responsible for any injury done by or to the contractor's employees from any source or cause.

39. **Removal of Rubbish, etc.**—After the completion of the work the contractor shall clean and sweep out the rooms and cellar and remove all rubbish from the grounds.

40. **Inspection.**—The work and materials shall, at all times, be open to the inspection of the Engineer, and all facilities must be afforded for examining the same.

Work or materials rejected for want of conformity with the plans and specifications must be removed and replaced with acceptable work and material at the expense of the contractor.

41. **Decision of the Engineer.**—All questions arising out of this contract shall be decided by the Engineer Officer in charge of the work, and his decision shall be final and conclusive.

Wherever the word "Engineer" is used in these specifications it refers to the Engineer Officer or his authorized agents, acting under his direction.

42. **Contract Time.**—The work must be commenced within......days after receipt of notification of approval of contract, and finished not later than........... The order of the work shall be as directed by the Engineer.

43. **Work on Sundays, etc.**—No work shall be done on Sundays or legal national holidays except in cases of extraordinary emergency, and only then by special permission of the Engineer.

44. **Payment.**—Payment will be made in one sum when all the work has been inspected and accepted. The fence will be measured for payment by the lineal rod, in position, complete, each gate being estimated as an equal length of plain fence.

GENERAL SPECIFICATIONS FOR A LOCK,

INCLUDING SPECIFICATIONS FOR THE FOLLOWING:

COFFER-DAMS; EXCAVATION; ROCK FOUNDATIONS; PILE FOUNDATIONS; CONCRETE MASONRY; STONE MASONRY; IRON-WORK. TIMBER GUIDE-CRIBS; TIMBER LOCK GATES; STEEL LOCK GATES;

SPECIAL DESCRIPTION.

1. The contractor must begin work within 30 days after receipt of notification of approval of his contract. He shall notify the Engineer Officer at least one week in advance of the day on which he intends to begin work.

2. In case the available funds shall become exhausted before the completion of the contract, the Engineer Officer will give 30 days' written notice to the contractor that work may be suspended; but, if the contractor so elect, he may continue work under the conditions of these specifications after the time set by such notice, so long as funds are available for expenses of superintendence and inspection, with the understanding, however, that no payment will be made for such work until additional funds shall have been provided in sufficient amount. In case of suspension due to lack of funds, work must be resumed within 30 days after receipt of written notice that the additional funds are available for continuing the work.

3. In case other contracts shall be in force at the locality during the progress of the work under these specifications, all roads and grounds, and the river in the neighborhood of the work, shall be open alike to the use of all the contractors, and the contractor for one part of the work will not be permitted so to arrange his plant or working force as to interfere with other parts of the construction.

4. **Contract Time.**—The contract must be completed within....months* after receipt by the contractor of notification of approval of his contract by the Chief of Engineers; provided, however, that in case of any interruption of the work due to failure of appropriations, the time fixed for completion shall be extended an equivalent period.

5. **Work to be Done.**—The work to be done under these specifications is the excavation for, and the construction, complete, of a lock and approach cribs, to be known as Lock No...., ...River. The location, dimensions, and general construction are to be as herein described, and as shown on maps and drawings to be seen at the U. S. Engineer office at................., which maps and drawings form part of these specifications.

6. **Location.**—The site for the lock is at.............., on the............Railroad.

7. **Dimensions and Material.**—The lock will be.....feet long over all, with a clear width of.....feet in the chamber; the walls will be of......., the gates of........ The construction of the cribs will be as hereinafter described.

---

* This may be varied to specify a fixed number of fair working days, allowing for inclement weather, highwater, etc.

8. Foundation.—The lock will be built on............................... ...., at an average depth of......feet below the lower miter-sill level. The bottom of the construction is shown on the drawings as....feet below the lower miter-sill level: this must be understood to be only approximate, as the character of the material encountered, or other circumstances, may necessitate deeper or shallower foundations.

9. Contract to Include.—Any necessary cutting away of the bank above the upper or below the lower cribs, to make the upper and lower approaches, will be done by the United States and the cement will be furnished by the United States. With these exceptions all material and labor, permanent or temporary, necessary to the entire completion of the work, according to the true intent and meaning of these specifications, will be included in this contract. The contractor, under his contract price, must furnish and pay for all material of every description entering into or connected with the temporary or permanent construction. He must also, under his contract price, supply and pay for all labor, skilled or otherwise, required to grub and clear the site of the lock and cribs, to prepare and place all materials, and to remove all débris after completion of the work, and must furnish all necessary plant, tools, landings, derricks, machinery, boats, and temporary buildings.

10. Details.—The details of the work shall conform to the drawings exhibited, and to such others in explanation of details or modifications of plans as may be furnished from time to time during the construction. Both the drawings exhibited and those that may be furnished are to be considered as forming a part of these specifications. These specifications are intended to be full and complete; any doubt as to their meaning or any obscurity in the wording of them will be explained by the Engineer, who shall have the right to correct any error or omission, whenever such correction is necessary for the proper fulfillment of their intention.

11. Investigation.—It is expected that each person bidding will visit the site of the lock and the office of the local engineer, and will ascertain the nature and location of the ground, etc., the general character of the river as to floods and low water, and will obtain all information necessary to enable him to make an intelligent proposal. Information given bidders as to quarries will not be considered to bind the United States to accept the output of such quarries. The quality of the material offered, and not its source, will be the basis upon which it will be inspected and judged.

COFFER-DAM.

12. Plan.—The United States will furnish plans for the coffer-dam, and it must be built in accordance therewith, unless minor modifications are desired by the contractor or by the United States, and approved by the Engineer Officer. Its height shall be as shown on the drawings. The foundation will be either on the rock or on the overlying material, at the discretion of the Engineer. If founded on the rock, the material above must be cleanly removed by dredging or otherwise. The ends of the coffer-dam must be securely rooted in the bank, and protected with riprap as directed.

If found necessary triple-lap sheet piling of 2″×12″ plank shall be driven from what is shown on the drawings as the approximate end of the coffer-dam to such point in the bank and to such depth as the Engineer may direct. Sluiceways shall be constructed as indicated on the drawings.

13. **Details of Construction.**—The timber and lumber used in the coffer-dam must be sound and sawed to full dimensions, but may be of any kind. The cribs must be sunk progressively, and as soon as any portion of the excavation is ready. They must be filled to full height with heavy material acceptable to the Engineer, and shall be banked on the outside, as directed by the Engineer, with material not liable to be washed away and subject to the approval of the Engineer; and riprapped where required; and shall be made sufficiently water-tight to prevent objectionable leakage. This filling, banking, and riprapping must be commenced as soon as any portion of the coffer-dam is in place, and must be carried to completion without delay.

If the contractor so desire, an opening, subject to the approval of the Engineer, may be left temporarily in the down-stream arm of the coffer-dam, to assist the excavation.

(*Note.*—If the coffer-dam is to be of the pile type, the following specification may be substituted.

The piles may be of any good live timber which can be driven without splitting. They shall be not less than 10 inches across at the small end, and sound and straight. They shall be driven to bed-rock and cut off at given elevations, and must be kept plumb and in line, and at the proper distances apart. After they have been driven they must be dapped and brought to fixed lines for the walings and ties, as shown on the drawings.

The sheet-piling must be driven close and plumb, and to bed-rock, and be cut off at the proper elevation and spiked to the walings. Obstructions to driving must be removed as far as practicable.

Sluiceways shall be constructed as indicated on the drawings.

The ends of the coffer-dam must be securely rooted in the bank, and banked and riprapped as directed.)

14. **Estimates and Payment.**—All bolts, spikes, and nails used in building any part of the coffer-dam must be supplied by the contractor, and covered in his general price for "Coffer-dam timber." All timber and lumber, including sheet piling, used in the coffer-dam will be classified as "Coffer-dam timber." The timber and lumber will be estimated as the lengths specified on the drawings or required in order to conform thereto, except that no deduction will be made for the parts necessarily cut out in making joints.* All material, except that ordered as riprap, used in filling

* In some contracts the United States has furnished bills of the lumber and piling to be used in the coffer-dam and elsewhere. In such cases payment is made by the dimensions billed, irrespective of cut-offs. It is advisable, however, where such bills are supplied, to make some provision for payment of material ordered but not used, as circumstances may arise during construction which will modify the amount required.

and banking the coffer-dam to the lines and levels established by the Engineer, will be classified as "Coffer-dam filling." The material used in filling will be measured in place, but without deduction for the space occupied by the cross-ties and cross-walls. The material used in banking will be measured by cross-section, in place, but none placed outside the established lines and slopes will be paid for. The riprap will be paid for as "Riprap," by the cubic yard in place. The material excavated for the foundation and shore connections of the coffer-dam will be classified as "Excavation," and will be estimated for payment by cross-sections as provided for in par. 22.

All labor and material not mentioned, but required in the construction of the coffer-dam, shall be furnished by the contractor at his own expense and covered by his prices on other parts of the work.

Payment for the coffer-dam, subject to the ten per cent deduction,* will be made on monthly estimates, but no payment will be made for any material lost or damaged during construction through the fault or negligence of the contractor, or through his failure to properly complete, fill, bank, or riprap the coffer-dam or any portion of it after having commenced work thereon, except where such failure arises from natural causes. All repairs or rebuilding, the necessity for which shall arise from natural causes, and which shall be ordered by the Engineer, will be paid for at the contract prices for the material used, estimated as provided in these specifications for the original construction: provided, however, that all repairs or work of any kind, rendered necessary in any manner through the fault or negligence of the contractor, or done for his convenience by permission of the Engineer, shall be executed by the contractor at his own expense.

15. **Removal.**—The contractor will be required to remove all material forming the down-stream arm of the coffer-dam to a depth of 1 foot below the lower miter-sill, except where it will be covered behind the lower land cribs. He must also remove all material forming the river arm below the dam to a depth of 2 feet below lower pool level, and all material forming the upper arm and the river arm above the dam that may come within 10 feet of the upper pool level, together with all portions that may interfere with any of the construction. The material removed need not be preserved. All this work must be done at his own expense, and the time of removal and the place of deposit for waste material shall be prescribed by the Engineer.

16. **Cement-testing Shed.**—A shed, for the purpose of testing cement, and a cement storehouse, shall be built by the contractor according to the drawings. The construction of these buildings shall be completed as soon as practicable after beginning work, and at least two months before the contractor will be in readiness to use cement. Any necessary repairs to these buildings during the continuance of the contract shall be executed by the contractor at the request of the Engineer. The lumber entering into their construction and their repairs will be estimated and paid for as "Coffer-dam

* See paragraph 30, under "General Conditions."

timber." The remaining materials used shall be furnished by the contractor at his own cost and covered by his price for "Coffer-dam timber."

17. Ownership.— The payments made for the different parts of the coffer-dam and cement-testing shed and the cement storehouse shall cover the entire cost thereof to the United States, and by virtue thereof they shall become the property of the United States.

18. Floods.—Whenever the river rises to a stage which, in the opinion of the Engineer, endangers the safety of the coffer-dam, the inclosure shall be filled through the sluiceways. The cost of all pumping shall be borne by the contractor.

EXCAVATION, EMBANKMENT, AND PAVING.

19. Clearing Site.—The removal from the site for the lock, cribs, paving, and coffer-dam, before and during construction of all trees, bushes, stumps, logs, roots, drift, and other rubbish, shall be performed by the contractor at his own expense, and the cost shall be included in the price paid for Excavation.

20. Classification.*— Excavation will be classified as "Rock Excavation," "Earth Excavation," and "Deposit," and will all be measured by contents in place, before removal. Rock which cannot be loosened by pick or bar, but requires blasting for its removal, and also all boulders of a volume greater than 9 cubic feet, will be classified as "Rock Excavation." Rock that can be loosened by pick or bar, and boulders having a volume of 9 cubic feet or less, and all other materials of whatever nature, except "Deposit," will be classified as "Earth Excavation." Material washed or left in the coffer-dam inclosures by floods, except that which slides in from the bank, will be classified as "Deposit," and the price paid for "Deposit" shall cover the cost of all necessary cleaning and scrubbing. The Engineer will give directions as to what part or how much of such deposit shall be removed. In general, deposit will not be ordered for removal, unless it interfere with the work, nor will any payment be made for deposit removed by the contractor without orders from the Engineer. The deposit or earth excavation which may be removed by the contractor in tearing out the coffer-dam will not be estimated for payment. No payment will be made for removing material washed into the inclosures from the coffer-dam, or from any dump made by the contractor on or above the work, nor for duplicating foundation excavation in cases where the hole has filled again because the masonry construction did not promptly follow the excavation, of which lack of promptness the Engineer shall be the judge. The prices paid for all excavation shall include the removal of the material to its place of deposit.

---

* The classification of "Deposit," especially where the banks are soft, results usually in uncertainty and dispute as to what part of the excavation actually was deposit and what part earth For this reason it will be found advisable, except in a few rare cases, to omit it entirely, and to include it in "Earth Excavation." For similar reasons it is usually best to omit "Loose Rock" classification, as where the stones are imbedded in earth, it is usually impossible to obtain an accurate estimate for each material.

21. **Removal of Bed-rock.**— In removing bed-rock by blasting care shall be used not to shatter the rock where any superstructure is to be laid upon it. No heavy charges shall be exploded in the bed-rock within 2 feet of the final surface, nor within 10 feet of any masonry; the final portions shall be removed by light blasts and picks and bars.

22. **Extent of Excavation.**—Preliminary lines, slopes, and grades will be given by the Engineer for all excavation, but they may be changed by him from time to time, as the nature of the work may demand; these lines, slopes, and grades will be the limits beyond which the contractor must not go. It must be understood, however, that the excavation will not necessarily be made to these limits, unless it is specifically ordered so by the Engineer, and that the slope of any excavation shall be as much steeper than the marked slope as the material will stand. The contractor will be paid only for the material actually removed. Any slip occurring through the negligence, fault, or delay of the contractor (of which negligence, fault, or delay the Engineer shall be the judge) must be removed and back-filled by him at his own expense if so ordered by the Engineer; but any slip occurring without negligence, fault, or delay on the part of the contractor will be estimated for payment if ordered by the Engineer to be removed. Any material excavated in the absence of, or in disregard of instructions from, the Engineer, or removed from outside of the lines, slopes, or grades given by him, will not be paid for, and must be back-filled by the contractor at his own expense, if so ordered.

In case any of the material to be excavated is removed and carried away by floods during the execution of the contract, the contractor will receive pay only for so much of it as may have been included in the limits of the latest slopes given him by the Engineer, except that if any of the space is filled up again, or partly filled up by deposit from the river, the contractor will not be paid both for the original material and for that deposited by the river in its place, but only for so much of that deposited as may be ordered to be removed.

The extent to which the contractor will be allowed to remove the bank inside the coffer-dam by dredging, and the time for commencing such work, will be subject to the orders of the Engineer, and if work be started too late in the season to permit laying sufficient masonry no dredging will be permitted. If it be started in time to build up most of the foundation, dredging will be permitted for so much of the foundations as may be deemed advisable, commencing with the river-wall. Should the contractor desire to tear out a certain portion of the coffer-dam in the following season, subject to the approval of the Engineer, in order to continue dredging, he will be allowed to do so, but such tearing-out, and the rebuilding, refilling, and backing of the coffer-dam to the original lines shall be at his own expense.

No dredging for the land-wall will be permitted beyond a line distant 10 feet from the chamber face of that wall, measured toward the bank. The remainder of the excavation must be done in the dry, after the coffer-dam has been pumped out.

Material in the chamber and entrances will not be removed except as may be

necessary for the construction of the masonry and of the approaches, and for proper facility of navigation.

Measurement for excavation will be made by cross-sections of the bank and river-bed taken just before beginning the work and at such times thereafter as may be necessary.

23. **Disposal.**—Excavated material shall be deposited as directed and where directed by the Engineer, and in such manner as not to interfere with present or proposed navigation. Material of any kind deposited by the contractor in absence of, or in disregard of, the instructions of the Engineer, shall, if so required by the Engineer, be removed and re-deposited by the contractor at his own expense. If the contractor desire to save any part of the excavated material for later use as embankment, the place of its deposit must be subject to the approval of the Engineer.

24. **Shoring.**—If deemed necessary for any excavation, it shall be securely shored and curbed by the contractor, with timber and plank, as directed by the Engineer, and shall be maintained by the contractor in this condition until the foundation shall have advanced sufficiently to allow the shoring and curbing to be removed. The piling and lumber thus used will be classified and paid for as "Piling" and as "Coffer-dam Timber" respectively, and shall become the property of the United States. After the necessity shall have ended, the material shall, in the discretion of the Engineer, be allowed to remain in place, or shall be removed and piled by the contractor, in such manner and in such place as may be directed, without extra expense to the United States. If required to do so, the contractor shall use, in any other curbing and shoring, such of this material as may be judged suitable by the Engineer, and for such use, including the labor of erection, with all spikes, bolts, etc., he shall receive 50 per cent of his price bid for similar new material. All nails, spikes, bolts, and other fastenings employed in the shoring and curbing shall be furnished by the contractor without extra cost to the United States, and shall pass into the possession of the United States with the lumber.

25. **Embankment.**—This will comprise the filling behind the lock-wall and the land-cribs, and any filling required in grading the banks which shall be specifically ordered as "Embankment" by the Engineer. All material used as embankment must be satisfactory to the Engineer. Material, to such an extent and in such locations as the Engineer shall direct, may be taken by the contractor from the banks of the river on the United States land. The cost of excavating the material used will be included in the price of "Embankment," except as it may already have been estimated and paid for as "excavation" and saved by the contractor for subsequent use as embankment, in which case the total payment for it will include the classification as "Excavation," and also as "Embankment." Embankment shall be commenced, carried on, and suspended at such stages of the construction as the Engineer shall direct, but no embankment shall be placed against masonry just completed. The grades for embankments cannot be given exactly until after all excavation has been made. The contractor shall have the privilege of using as embankment such of the excavated material

as, in the opinion of the Engineer, shall be suitable for this purpose. Where so directed, the material shall be moistened and rammed in place as the filling progresses.

The embankment or back fill behind the land-wall of the lock on which the paving is to be placed must be made during the first working season to the height of the land-wall as then built, if any of the latter is in place. If deemed necessary by the Engineer, riprap shall be placed on the fill to a depth of about 1 foot to protect it during the winter and spring rises, payment being made for this as "Riprap."

The paving itself shall in no case be placed until the completed embankment and back fill have been subject to the exposure of one winter's floods and spring rains. Any additional filling to be made after such exposure shall be thoroughly well rammed to the satisfaction of the Engineer.

To allow for shrinkage the final measurement will not be made until two months after the completion of the embankment.

26. **Paving.***—The space behind the land-wall and between the wing-walls, to a distance of about.... feet from the chamber face shall be covered with stone paving, with a curb wall 12″ wide along the back and at each end, of a depth of.... feet. Every stone so used shall have upper surface dimensions of at least........... The stone shall be sound, and well shaped, laid with joints not exceeding ¾ of an inch, and all joints must be grouted with three to one cement mortar to a depth of 2 inches. No variation of more than 1 inch from the grade line will be permitted. The stones shall have a depth of 10 to 12 inches, and shall be placed on a bed of spalls or river gravel 6 inches in depth, laid on the graded and rammed surface of the earth. The entire surface must be set to the lines and slopes given by the Engineer. The price bid per square yard for paving must include the cost of furnishing and placing the bed of spalls or gravel upon which it is laid, and the grouting of the joints. None of this paving and curbing shall be laid until the embankment shall have settled sufficiently for the purpose, as specified in the preceding paragraph. The paving and curb-walls will be estimated per square yard.

(*Note.* — If concrete paving is to be used in place of the above, which is common or rough-dressed paving, the following may be substituted for the first few sentences.

Paving shall be of cement concrete, well-rammed and faced as described for the coping. It shall be 6″ in total thickness, and shall be similarly laid by experienced sidewalk masons and be divided into rectangular blocks as shown on the drawings. Curb-walls to be paid for as "Paving," and of the same material as the paving, 12 inches wide and 2 feet deep, with rounded top, 2 inches above the paving, shall be laid along the back edge of the paving and along the sides to meet the wing-walls. Before laying the paving and curb-walls, the earth embankment underneath shall be rammed and graded carefully to 6 inches below the bottom of the concrete, then a bed of spalls or gravel shall be laid, rammed, and graded on the earth, of sufficient depth

* This clause may be extended to include paving the banks above and below the land-wall, where such is desired.

to bring its top to the level of the bottom of the concrete. The price bid per square yard for this paving must include the cost of furnishing and placing the bed of spalls or gravel upon which it is laid.)

27. **Cement Sidewalk.** — Before the completion of the paving a cement walk 3½ feet wide shall be built from the back of the slope paving to the back of the land-wall, with steps where required. For this walk a trench shall be excavated 12 inches deep below the finished surface of the paving; the bottom shall be well rammed, then covered with spalls or gravel also well rammed, up to 6 inches below the finished grade; this stone or gravel shall then be covered with 4 inches of cement concrete, of the same composition as that in the lock-walls, well rammed and topped with cement mortar, two parts of sand to one of cement, to the required grade, all divided into blocks 3 feet long, and finished in the most approved manner by skilled sidewalk finishers. The sidewalk and steps will be paid for as "Paving," only the horizontal surfaces of the treads being estimated for payment; the layer of broken stone or gravel will not be paid for, but must be included by the contractor in his general price for "Paving."

28. **Puddling.**— An embankment of heavy moist clay, thoroughly cut, worked, and rammed in 6-inch horizontal layers, shall be placed at the end of and on both sides of the upper wing-wall of the lock, as directed by the Engineer. This clay will be classified as "Puddling."

FOUNDATIONS.

29. **Changes or Modifications.**—The character and positions of the proposed foundations for the different parts of the work are shown in general on the drawings and cross-sections; the United States shall have the power to make any changes in the plans of the foundations which, in the judgment of the Engineer, may be considered advisable, after examinations made during or after excavation, and the contractor shall have or make no claim against the United States on account of any such changes or modifications, or on account of any increase or decrease in the depth of foundations from those referred to herein or shown on the drawings, other than that for payment for the actual amount of work ordered by the Engineer, and completed at the unit price bid.

30. **Cleaning for Foundations.** — All rock surfaces for foundations must be freed from loose pieces and be worked down to the firm and solid rock, and be cut out and roughened as required by the Engineer, to give good bond for the masonry. The foundations shall be thoroughly scrubbed and then washed clean by jets of water under heavy pressure before any masonry is laid. Laying masonry will not be permitted on any surface that is not thoroughly clean. Any seams or crevices appearing in the rock must be scraped out and filled to the satisfaction of the Engineer, with concrete or mortar thoroughly rammed or worked in.

(*Note.*—If the foundation is to be of piles, the following specification may be used: The piles must be of hardwood, not less than 10 inches in diameter at the small end, nor less than 14 inches at the butt. They must be sound, straight, and cut from live trees, and have all bark removed. They shall be driven to bed-rock, at the designated distances apart, and shall be framed, capped, and floored in a workmanlike manner as shown on the drawings. Where the heads are to be imbedded in concrete the tops shall be sawed off square at the required height, and the surrounding material shall be excavated to grade and heavily rammed where practicable. Broomed or injured pile-heads will not be permitted in the finished work.

Sheet-piling shall be of sound, well-sawed, white-oak plank, free from defects liable to injure its durability, and must be carefully driven so as to form a wall as tight as practicable. Any pile that shall be injured in driving must be replaced with a sound one to the satisfaction of the Engineer, and without expense to the United States. This piling must be driven down to.........and be cut off as shown on the drawings.

Payment for the round piles will be made by the lineal foot, and for the sheet-piling, caps, and flooring, per thousand feet board measure, all lengths being taken as those of the pieces specified on the drawings or required to conform thereto.*

Round piles will be classified as "Piling," and sheet-piling, caps, and flooring, as "Timber in Permanent Construction.")

CONCRETE.

31. **Proportions.**—All concrete shall be made with Portland cement. The proportion of the ingredients shall in general be one part of cement,......parts of sand, and ......parts of gravel or broken stone, all measured by loose volume.† Accurate methods of measurement must be employed, with boxes or otherwise, so as to secure the proper proportions for each batch, but the proportion must be varied when so directed by the Engineer.

32. **Cement.**—The cement will be furnished in cloth sacks‡ by the United States, and will be delivered to the contractor on the cars at.................., from which place he shall transport it without delay to the cement house, and store it there, each car-load being kept separate, to a height not to exceed 5½ feet. It will be issued to him from the shed at such times and in such quantities as the needs of the work may require. The cost of all hauling and handling must be included in his price bid for "Concrete." He will be held responsible for the cement after he has been notified of its arrival, until he has placed it in the storehouse, and must protect it from deterioration. Demurrage charges, occurring through the negligence or fault of the contractor, together with the cost of any cement, or any sacks lost or damaged while in his hands,

---

* See note to paragraph 14.

† See Part III. Chap. II., "Proportions."

‡ Paper bags are only satisfactory when cement is to be used as fast as delivered and when the distance shipped is short. Barrels are often specified when cement is to be held over winter in a damp climate.

either before storage or afterward, will be deducted from any sums due or to become due him. The United States will use all possible means to keep a supply of cement on hand, sufficient to prevent any delay in the progress of the work, but should such delay occur from any cause, no claim for damages caused by such delay shall be made by the contractor against the United States. The contractor must collect, bale up, and return to the railway station all the sacks, free of expense to the United States.

(*Note*.—If the contractor is to supply the cement, the specification may be changed to the following:

All cement must be supplied by the contractor, and may be of the following standard brands............... The use of more than one brand in the same wall will not be permitted above the height of.......feet above the foundation. The contractor must protect the cement from injury while in the cement shed or in handling, as any cement wetted or otherwise damaged will be rejected. He shall permit access to it at all times by the Engineer or his inspectors, and shall afford facilities for taking out samples. Each car-load must be piled separately, so that any rejected lot may be easily recognized.

The cement shall be supplied in cloth sacks, four sacks being equal to one barrel, and one barrel to weigh 375 lbs. net. If the weights as determined by test weighing are found to be below this, the deficiency must be made up without expense to the United States. All packages must be plainly marked with the brand, and any not so marked will be rejected.

The cement must fulfill the following requirements, tests for which will be made by the United States at the lock site, and no cement will be accepted for use which has not fulfilled them. [See specifications for cement, p. 335, et seq.]

33. **Sand.**—The sand shall be clean and sharp, free from all earth and vegetable matter, screened and washed if required, and satisfactory to the Engineer in every respect; it shall be kept free from dirt, and stored until well drained. The use of wet sand will not be permitted.

The sand must fulfill the following specifications for coarseness of grain:

To pass through a No. 50 sieve, not more than 50 per cent; to pass through a No. 100 sieve, not more than 2 per cent.

34. **Broken Stone.**—The broken stone shall be hard, sound, and entirely clean. It must range in size from pieces $\frac{1}{10}$ of an inch to pieces $1\frac{1}{2}$ inches in diameter.

The contractor must supply samples of the stone he intends to use, and if such samples are accepted all the stone used must be of equal quality. Accepted stone shall be stored on board platforms, if so directed by the Engineer, and be kept clean.

35. **Gravel.**—Gravel shall be clean, thoroughly washed, and free from shale and other foreign matter; it must range in size from pieces passing a $1\frac{1}{2}$-inch screen to pieces retained on a $\frac{1}{10}$-inch screen, but may contain sand not to exceed 6 per cent of its volume. Accepted gravel shall be stored on board platforms, if so directed by the Engineer, and be kept clean.

36 Forms.—The forms in which the concrete is to be placed must be built in accordance with the drawings, and be solidly braced and held at all times rigidly in position so as to secure accurately the required outlines of the masonry. No bracing will be permitted to rest against any piles, trestles, track, or other support liable to yielding or to vibration, and any deviation of the forms from line before or during completing must be at once corrected. No concrete shall be laid in any section until all the posts have been lined up, braced, and partly planked to the satisfaction of the Engineer.

All form lumber shall be of long-leaf yellow-pine, of good quality, sound and well-seasoned, and free from defects which would disfigure the concrete. The use of warped or crooked lumber will not be permitted. The edges of all posts shall be dressed, and all plank or lagging against which any facing mortar is to be placed, as hereinafter specified, shall be dressed on all sides, and be of even width. Especial care must be taken to secure smooth and even surfaces in the concrete, and all dressed lagging must be neatly fitted and joined, and be shimmed where required, and must be laid and kept level, with joints on the same line as the joints of the adjoining section. Where the butt joints do not come on posts, they must be satisfactorily reinforced.

Lagging shall be of......by......lumber, before dressing.*

Posts shall be of 3-inch by 8-inch lumber, before dressing, spaced not more than ....feet apart, with braces at top and bottom, and intermediate braces where required.* Braces on the same post shall not be further apart than 7 feet, measured along the post. They shall be of the following sizes:

For lengths of 10 feet or less, of 2-inch by 4-inch lumber, or larger,
" " " 10 feet to 16 feet, of 3-inch by 4-inch " " "
" " " 16 " to 22 " of 3-inch by 6-inch " " "
" " " 22 " to 26 " of 4-inch by 6-inch " " "
" " above 26 feet, of 4-inch by 6-inch lumber, shored and stiffened where so directed.

They must be provided with heels or bases of ample size, well rammed to a solid bearing where required, and be secured to the posts with blocks or otherwise so that no slipping can occur. Where so directed by the Engineer, additional braces and shores for braces must be provided. Diagonal bracing of plank not less than 6 inches wide and 1 inch in thickness must also be provided to hold the posts rigid in a lateral direction. All braces must be set as nearly as practicable at an angle of forty-five degrees or less to the horizontal; where this angle is exceeded they must be reinforced, or the heads of the posts must be tied to those of the opposite posts, as may be directed. Where the braces support posts for battered surfaces, the angle must be proportionately reduced.

If tie-rods are used instead of, or in addition to, braces, they must be not less than

* See "Forms," Part III., Chap. II.

$\frac{5}{8}$ inch in diameter, and where two or more are used in the same post they must not be further apart than 7 feet vertically. They shall be provided with sleeves about 2 feet from each end, so that these ends can be withdrawn when the concrete is finished. Bolts built into the wall for the purpose of holding any portion of the forms shall be not less than $\frac{3}{4}$ inch in diameter, and these and the ends of the tie-rods must be greased before being built in. No iron or wire used in fastening the forms and left imbedded in the concrete must come within 4 inches of the face.

Facing templates shall be of dressed plank 8 inches wide, 2 inches thick on the top edge, and 1 inch thick on the bottom edge, and shall be provided with rings or staples for lifting. They must be lifted up carefully, and without wrenching or prying, so as not to disturb the adjoining facing, and must be kept in good working condition.

Forms shall remain in position for at least three days after the concrete has been completed in them; and must remain longer in cold weather. They shall be removed by the contractor at his own expense, and such materials in them as may be satisfactory to the Engineer may be used in the construction of other forms after proper cleaning and trimming.

All material used for the construction of forms must be supplied and placed by the contractor at his own expense, and be included in his price for concrete.

37. **Voids and Openings.**—All necessary voids and openings for conduits or wells, or for such other purposes as may be required by the Engineer, shall be left in the concrete. Recesses for ladders, coffer-beams, etc., shall be formed by wooden boxes of the required shape, dressed to leave a smooth surface on the finished work.

38. **Mixing and Placing.**—The concrete is to be mixed by machinery except when the quantity required is very small, when it may be mixed by hand. The gravel or broken stone shall be thoroughly drenched just before being put in the mixer, and the order of placing in the mixer shall be as directed by the Engineer. The water used in the concrete must be clean, and the proportion regulated as directed, so as to produce a concrete which will quake after being rammed. All mixing must be thoroughly done and must be satisfactory to the Engineer, and shall be continued until every particle is covered with mortar; the mass shall then be kept in motion so that its initial set will not occur before it is in its place of deposit. All mortar, in concrete or elsewhere, must be used before the cement has begun to set, otherwise the mortar must be wasted and the cement will be charged against the contractor as damaged cement. No re-tempering will be allowed under any circumstances. If the water to be used is taken from the river, the contractor must provide settling tanks to the satisfaction of the Engineer, to be used whenever the river water is too muddy to be fit, in his judgment, for mixing mortar or concrete.

Before beginning any foundation section of concrete, the bed-rock shall be thoroughly cleaned, the rock being roughened, if so directed by the Engineer, as described in paragraph 30, and a grout of neat cement shall be spread thereon and thoroughly brushed in, to provide a tight joint between the foundation and the wall.

As soon as concrete is deposited in the forms it shall be spread in layers not more than 8 inches thick and be compacted by ramming. The engineer shall prescribe the number of men to be employed on this part of the work, as the ramming must be thorough. The rammers shall be of iron or steel, with flat rectangular faces 6 inches square, weighing with the handle about 20 pounds each. When deficiency of moisture is apparent after the ramming has been completed, water must be supplied by sprinkling, and all exposed surfaces of unfinished work must be kept constantly moist by sprinkling, at short intervals, if so directed, until thoroughly set.

The concrete above the lower miter-sill level must be built in blocks of the dimensions shown on the drawings, arranged in regular courses for the full length of the wall. Each horizontal course of blocks shall be of the same height throughout, and the vertical joints shall be continuous. The ends of adjacent blocks and the surfaces of consecutive courses shall be well bonded into each other. On those faces of the blocks which are to be formed against dressed lumber the contractor will be required to mark out the joints between adjacent blocks, by dressed battens nailed against the inside of the forms, the battens to be of dimensions approved by the Engineer.

(*Note.*—If the wall is to be built in monoliths of the full height, the following clause may be substituted for the preceding:

The concrete above the foundations shall be built in monoliths extending from the foundation course to the top of the wall, provision being made for the proper bonding of the adjacent blocks. The concreting shall be carried on in daytime only, but work once commenced in a block must be prosecuted each succeeding day without interruption other than that caused by suspension of work for Sundays or legal holidays.)

Whenever concrete is to be placed on a block or a layer that has set, all chipped or broken edges shall be cut out, the surface shall be thoroughly brushed off and wetted, and shall be covered with a grout of neat cement worked into the surface with brooms. No concrete will be allowed to come into contact with a dry, dirty, or dusty surface.

Whenever concreting is suspended for more than one hour on any block, course, or layer, the outer edges shall be brought to a level, and where facing is used they shall be carefully struck off with a trowel and a straight-edge before the cement has begun to set. No uneven edges will be allowed, and the concrete must be kept approximately level during the laying. Whenever work is thus suspended, the center of the last layer shall be left as a ridge about 6 inches higher than the outsides, so as to provide a bond for the next layer, and all the concrete in this layer, together with its facing, shall be mixed with an additional amount of water, to prevent drying out.

No concrete shall be laid in water except to stop leaks or springs, nor exposed to the action of running water until thoroughly set.

The use of slides or chutes for depositing concrete will not be permitted.

No concrete shall be laid at night, unless specially ordered by the Engineer.

39. **Facing.**—All surfaces of the walls which will be visible after the work has been completed, except the coping, together with all portions for a width of 2 feet lying imme-

diately below the bottom lines of those surfaces, shall have a facing averaging $1\frac{1}{2}$ inches in thickness, composed of one part of cement and two parts of sand thoroughly rammed in layers not over 4 inches deep, with a special rammer. This rammer shall be of bar iron, 1 inch square and 6 inches long, with a bent gas-pipe handle, and shall have a total weight of about 8 pounds. The facing and backing shall go on simultaneously in the same horizontal layers. This work must be carefully and thoroughly done, and the contractor must provide competent laborers who shall be retained on the facing work whenever it is being carried out. No careless or unskillful laborer will be allowed to work upon it. If the mortar is mixed in the concrete mixer, the box must first be thoroughly scraped free from particles of stone or gravel, and no stone or gravel must show on a faced surface. As soon as the forms are removed, all such surfaces shall be examined, and all joint marks, lumps, or other disfigurements, shall be carefully effaced. Facing which is broken or otherwise injured at any time before the completion of the contract shall be cut out and satisfactorily replaced without cost to the United States.

Sand used for facing must be free from gravel, grit, or other material liable to show on the finished work.

40. **Coping.**—The coping on top of the lock-walls and wing-walls shall be not less than $\frac{1}{2}$ inch thick, and composed of one part of cement to two parts of sand, well bonded to the surface of the concrete below before the latter has attained its initial set. The entire coping layer shall be divided, as shown on the drawings, by joints about $\frac{1}{8}''$ wide, and the edges of the walls shall be rounded as shown on the drawings. All this work must be done by experienced sidewalk masons, and the contractor must remove and replace at his own expense any of it that is not of the best class, and done to the satisfaction of the Engineer.

Sand used for coping must be coarse-grained, but free from gravel and foreign substances.

All coping and facing will be estimated as concrete.

41. **Protection, etc., of Concrete.** — Whenever concreting is suspended the surface of the layer shall be at once completely covered by wet tarpaulins. These tarpaulins shall not be removed for two days, unless work is recommenced on the block before the two days have passed. Unfinished surfaces must be similarly protected from the effects of sun and wind whenever so directed by the Engineer, to prevent drying out before the mixture has set, and any concrete, facing, or coping, injured through lack of protection shall be at once removed and replaced at the contractor's expense.

All concrete shall be thoroughly drenched twice a day for three days after it has been deposited.

42. **Frost.**— No concrete shall be placed in temperature lower than thirty degrees Fahrenheit, nor when, in the opinion of the Engineer, it is liable to freeze before the mass shall have set sufficiently to prevent injury; the contractor will be required to protect at his own expense all work liable to be injured by the action of frost. In case

frost shall injure any work which has not been properly protected, or which has been placed in disregard or absence of instructions from the Engineer, the damaged work shall be torn out and replaced by the contractor at his own expense.

43. **Gauges.**—Recesses shall be left in the walls, as shown on the drawings, where tile gauges and tablets may be placed later. The tablets will be 4 feet square, the gauges 18 inches wide, one........feet, the other........feet long. These tablets and gauges will be furnished by the United States, but must be set by the contractor at such time as may be directed.

44. **Wing-walls.**— The wing-walls shall be founded on bed-rock for a distance of........feet into the bank from the chamber face of the land-wall; but, if later found desirable, their length may be increased or diminished.

45. **Measurement.**—Concrete will be paid for by the cubic yard. Only the actual amount of concrete in place will be estimated, deductions being made for all voids in the mass.

STONE-MASONRY.

If the masonry is to be of stone instead of concrete, the following clauses may be substituted. The faces of the chamber are supposed to be of "pointed face" stone.

(*a*) **Quality of Stone.**—All stone shall be perfectly strong, sound, hard, free from injurious seams, and in all respects satisfactory to the Engineer. It shall be of quality such as can be truly wrought to such lines and surfaces, whether plain or curved, as may be required, and shall weigh not less than 150 pounds per cubic foot. The United States reserves the right to reject any stone not deemed suitable. Stone quarried during freezing weather must have been seasoned for a period deemed sufficient by the Engineer before being laid in the wall.

(*b*) **Sample Cubes of Stone to Accompany Bid.**—Each bidder must deposit at this office two 6-inch cubical blocks of the stone he proposes to furnish, one of face stone and one of backing stone, with a statement giving the locality of the quarry from which the sample is procured, and the quality of all stone delivered under these specifications must be at least equal to that of the sample furnished. The sample must be truly squared on all sides, and dressed as follows: One side, smooth or rubbed finished; one side, fine pointed; one side, rock face; one side, bush-hammered; one side, rough pointed; one side, crandalled. A bid unaccompanied by a sample block of the stone offered, as above described, will not be considered. The bidder must satisfy the United States of his ability to furnish acceptable stone. A certified test of crushing strength and absorption must also be furnished if required.* Bidders will also state if their stone has been used before, and for how many years structures built of it, if any, have been satisfactorily standing. The contractor must also furnish from time to time, when so

---

* For a latitude such as that of the Ohio River valley the absorption of the stone should not be more than 2 or 2½ per cent; for warmer latitudes, where danger from frost is less, it may be limited to 3 per cent.

required, without expense to the United States, sample cubes from any portions of the quarry or quarries, shaped and dressed as may be directed by the Engineer.

(*c*) **Classification of Masonry.**—The masonry will be classified as "Hollow Quoins," "Special Stones," "Coping," "Pointed-face," "Rock-face," and "Backing." Hollow quoins, special stones, and coping will be paid for by the cube of the least rectangular figure that will contain the piece in question; the other classes will be paid for by the actual contents.

(*d*) **Hollow Quoins.**—Hollow quoins shall comprise the curved stones behind the gates. They shall be well and truly shaped from selected stone, in accordance with the detail drawings. The concave surfaces shall be neatly chiseled, together with such portions of the convex surfaces as may be directed; the remainder of the exposed surfaces shall be fine-pointed. The beds and vertical joints shall be dressed throughout without slack or want, and the stones must bond properly with the courses above and below, and be laid with ⅜-inch joints. They must be set square and plumb, and the contractor must remedy at his own expense any defects of setting.

(*e*) **Special Stones.**—Special stones shall comprise the top course of the upper and of the lower miter-wall where dressed to support the sills; the top courses of the upper and lower coffer-walls; recess stones for coffer-beams, ladders, and line hooks; recess quoins; end quoins; and pivot stones for gate pintles. No other stones will be classified as "special." The tops of all sill stones shall be fine-pointed; the rabbeted recesses shall be chiseled smooth; and the down-stream corners of the tops shall be chiseled to a quadrant of 2-inch radius. The exposed surfaces of other special stones shall be dressed similarly to the adjacent stone of the same course, unless otherwise directed. All masonry of this class shall have beds dressed entirely through, and the vertical joints shall be full for 12 inches from the face, and for as much more as the stone will allow. Special stone shall be laid with ⅜-inch joints.

(*f*) **Coping.**—Coping shall comprise the top courses of the main and wing-walls. It shall have all exposed faces fine-pointed, with a quadrant of 2-inch radius on the front edges where directed. The coping of the chamber and wing-walls shall be crowned ⅜ inch, and that of thicker portions of the walls shall be laid to a grade for drainage. It must all be of selected stone, with beds and vertical joints cut full and true throughout, and shall be laid with ⅜-inch joints. The upper and lower ends of the river-wall, for a distance of 20 feet, and such portions of the upper wing- and land-wall as may be directed, shall be doweled with 1¼-inch round iron, the dowels to extend through two and one-half courses. The iron shall be furnished by the contractor, and be set in neat cement, and the drilling and setting shall be carefully done as directed by the Engineer, and shall be paid for under the price for "Bolt-holes in Masonry."

(*g*) **Pointed-face Stone.**—Pointed-face stone shall be used for the general faces of the walls, and shall have its exposed surfaces pointed down fair and true, so that there shall be no projection above the face of the stone greater than ⅜ of an inch, and no depression below, the plane of the face being accurately determined by a pitch-line on

the exposed edges of the beds and vertical joints. The beds shall be cut entirely through and be parallel, and vertical joints shall be full for not less than 12 inches from the face, and for as much more as the stone will allow. Pointed-face stone shall be laid with ½-inch joints, except where it joins special or coping stone, when it shall have chiseled drafts and ⅜-inch joints at the abutting surfaces. It shall be furnished as headers and stretchers in the proportion of one header to two stretchers, and no stone shall be less than 4 feet in length or 2 feet in breadth, except by special permission. Stretchers must have one-third more bed than rise.

(*h*) **Rock-face Stone.**—Rock-face stone shall have the exposed faces accurately determined by pitch-lines around the edges of the beds and vertical joints, and there shall be no projections greater than 4 inches beyond these lines. The cutting and the dimensions of the joints, the sizes and the proportion of headers and stretchers, and the requirements for laying next to special or coping stones, shall be the same as for "Pointed-face Stone."

(*i*) **Backing.**—Backing shall be of well-shaped stones, of not less than 6 square feet of area on the smallest bed, with the sides hammered or picked off so that the vertical joints between backing stones will not exceed an average of 4 inches. Beds shall be full and approximately parallel, so that the bed-joints shall average 1 inch in thickness for three-quarters of the area of the stone. No part of the stone shall be higher than the face stone, and no through vertical joint will be allowed with face and backing stone, but special care must be taken to prevent the passage of water.

(*j*) **Courses.**—The number and arrangement of the courses shall be as shown on the drawings, and no variation therefrom will be allowed without special permission.

(*k*) **Position of the Various Classes of Stone.***—The foundation masonry, where exposed to the action of water, down stream from the upper hollow quoin, shall be faced with rock-face stone unless otherwise directed, and except where special stones are required. This shall include the lower miter-wall, the lower coffer-wall, the chamber faces of the main wall to the level of the top of the lower miter-sill (except in the gate recesses where it shall be one course lower), and the down-stream face of the upper miter-wall. Up stream from the upper hollow quoin the foundation masonry shall be of backing, except where special stones are required, and shall include the upper miter-wall, the upper coffer-wall, and the chamber faces of the main walls to the level of the top of the upper miter-sill, except in the gate recesses, where it shall be one course lower. The lower ends of both main walls, to a height of 2 feet below the lower pool level, and the outside of the river-wall to the same height, and the upper ends of both main walls to a height of 2 feet below the upper pool level, shall be of backing.

The inside faces of the main walls above the foundation masonry shall generally be of pointed-face stone.

---

*This description is for a lock with a fixed dam and a high upper miter-wall. In a lock with a movable dam the miter-walls are usually near the same level, and the outside of the river-wall requires pointed or smooth-face work, and a vertical face where the dam abuts against it.

The top courses of the upper and lower miter-walls on the down-stream side of the sills, and the top courses of the upper and lower coffer-walls, shall be of special stone, as directed in paragraph (*e*). These stones shall be pointed-face, with checks cut for the sills, and smooth chiseled.

The river face of the river-wall, beginning at a point 2 feet below the lower pool level, the lower ends of both main walls, and the exposed face of the lower wing-wall, beginning at the same level, and the upper ends of both main walls, and the exposed face of the upper wing-wall, beginning at a point 2 feet below the upper pool level, shall be faced with rock-face stone. That portion of the river wall against which the dam is to abut shall be picked off to the satisfaction of the Engineer without extra cost to the United States.

The inside of the culverts, except where special stone is to be used, shall be of rock-face stone.

(*l*) **Dressing, etc.**—The beds of all stone must be their natural quarry-beds. All stone must be shaped up before being brought on the walls, and the style of face-dressing must be uniform for all stones of similar finish. The edges of cut stone must be truly and sharply finished, and no stone of this class with chipped edges will be accepted. No lewis holes, dog holes, letters, or other disfigurements will be permitted on any face visible in the finished work. No stone finished by machinery will be permitted on any exposed face.

(*m*) **Laying Masonry.**—Before setting any stone the stone itself and its site shall be thoroughly wetted and scrubbed clean. Every stone shall be well laid to proper lines and in full beds of mortar, and be set with heavy wooden mauls, and must properly bond and break joints with the adjacent pieces. The bond of any stone shall in no case be less than 9 inches. The spaces between the joints of backing stone shall nowhere exceed 8 inches, and must within these limits be as small as the stone will allow. These spaces shall be filled with mortar, into which wet spalls must be well rammed in layers; the spalls must not be put in first. Backing shall not be laid in advance of face stone, and face stone must be promptly and thoroughly backed. No face stone shall be set until the stone below it has been thoroughly backed.

Not more than 3 unfinished courses will be allowed on any wall without permission of the Engineer.

No stone shall be dressed or hammered without permission after having been set in the wall. Any stone chipped or spalled after having been set shall be removed and replaced at the contractor's expense, or shall be paid for at a cheaper classification, at the option of the Engineer. Stones having defects purposely concealed will be rejected, whether set or not.

No joints in face stone will be allowed above or below a header.

(*n*) **Special Dimension Stone.**—The contractor, if so required by the Engineer, must get out additional stone to special dimensions and drawings, the finish of each stone to determine its classification and payment, as hereinbefore specified.

(*o*) **Cement and Sand.**—(See preceding clauses for Concrete.*)

(*p*) **Mortar.**—Mortar of natural cement shall be composed of two parts of sand to one part of loose cement; mortar of Portland cement shall be composed of three parts of sand to one part of loose cement, unless otherwise specified. No mortar of natural cement shall be used within 2 feet of the outside of the wall, except below the level of the lower miter-sill. All mortar must be mixed thoroughly and in small batches, and used before it has begun to set, and the mortar-beds must be kept covered when so directed by the Engineer.

(*q*) **Pointing.**—All exterior joints shall be scraped out as soon as filled, and shall be subsequently properly cleaned, wetted, and pointed, to a depth of not less than 1 inch with Portland cement mortar, composed of one part of cement to one part of sand, thoroughly hammered and finished with proper tools. Before the final acceptance of the work all such joints which have not been satisfactorily pointed shall be scraped out to a depth not less than 1 inch, and shall be repointed to the satisfaction of the Engineer without cost to the United States.

(*r*) **Sills, Anchorage of.**— The miter-sills and coffer-sills at the upper and lower ends of the lock are to be anchored to the masonry by bolts, as shown. The bolts shall be furnished and set by the contractor. The stones under the sills through which the anchor-bolts pass must be selected, cut, and set with a view to drilling for the bolts. Holes for the bolts must be carefully drilled in such a manner as not to jar or disturb the masonry through which they pass.

(*s*) **Pivot-stones.**—The four pivot stones, are to be selected stones, carefully cut and set according to directions given. The cast-iron pintle-plates shall be furnished by the contractor, and placed in mortar of neat Portland cement. The cost of cutting for and setting these plates will be covered by the price for "Special Stones," which will be the classification of these stones, as hereinbefore stated.

(*t*) **Gate Anchorages.**— The stones under the line of the gate anchorages, through which the anchor-bolts pass, must be selected, cut, and set with a view to drilling for the bolts, and extend to a depth of......feet below the top of the coping. They shall be laid in Portland cement mortar, and have no joints in the backing on the vertical line of the bolts. Holes for the bolts must be drilled in such a manner as not to jar or disturb the masonry through which they pass.

(*u*) **Frost.**—Masonry shall not be laid when the temperature is below 30 degrees Fahrenheit, except by special permission, nor when, in the judgment of the Engineer, it is likely to be injured by frost. All new or unfinished work must be protected from frost by the contractor at his own expense, and any work injured by lack of proper protection must be removed and replaced by him without cost to the United States.

---

* It will generally be found preferable to use a Portland cement throughout, although a good Eastern natural cement will give satisfactory results for the foundation and interior of the wall. The saving in cost, however, is small, especially as a richer mortar must be used with the latter kind.

(*v*) **Drilling for Bolts.**— The cost of drilling for bolts in the masonry, and of setting them, shall be covered by the contractor's price for "Bolt-holes in Masonry."

SILLS, CRIBS, AND LOCK-GATES.

45. **Classification.** — Timber in the lock-gates, in the miter-and coffer-sills, in the cribs, and in the sheet piling of the wing-walls (and in the foundations), will be classified as "Timber in Permanent Construction."

46. **Quality.**—All timber in permanent construction shall be sawed, and shall be of the full dimensions given, out of wind, free from large or loose knots, wind shakes, splits, wane, dote, or any defect tending to impair its strength or durability. All except that in the cribs and sheet-piling of the wing-walls must be dressed on all faces, and be of the full dimensions given after dressing; and shall be of............, except that sills must be of first quality white oak. All dressed timber must receive one coat of pure linseed oil on all sides and ends, as soon as dressed.

The timber may be inspected on arrival at the work, and rejected at that time if found to be defective; but this inspection will be only preliminary, and the final acceptance will not be made until the timber has been placed in the work.

47. **Miter-sills.**—The timbers for the miter-sills shall be selected sticks, neatly fitted in place and set and grouted where needed with neat cement, to make water-tight joints. They shall be secured with bolts as shown on the drawings and shall be set in perfectly true line and position. The faces of the sills against which the gates rest must be planed in place if necessary, so as to give a smooth and true fit.

48. **Cribs.**—There will be............cribs, of dimensions and position shown on the drawings. They shall be built of $10'' \times 10''$ timbers and filled with riprap.

All timbers in these cribs shall be fastened at each intersection and at each end with one drift-bolt, $\frac{5}{8}$ inch square and generally 17 inches long. Stringers shall be ....feet long, placed to break joints vertically and horizontally; all those in the chamber faces of cribs shall break joint over a tie, with square sawed ends; the intermediate and rear ones may break joint over a tie or midway between two ties; in the latter case a block 24 inches long shall be inserted under the joint, and two 28-inch drift-bolts driven through the top stringer and the block into the stringer below.

There shall be made and placed....fastenings for rafts, of design shown on drawing, all as directed by the Engineer. All drift-bolts shall have wedge points and countersunk heads, and the contractor shall submit a sample for approval before ordering them; they may be of either iron or steel. Holes shall be bored with $\frac{5}{8}$-inch augers.

49. **Ladders.**—Each crib shall have a recess cut for a ladder, and this ladder shall be securely spiked to the crib with 40^a wire nails. The ladder shall be of 2 by 4 inch white oak or yellow pine, with round rungs of 1-inch iron placed....inches apart; the width of the ladder inside shall be 14 inches and the length for the lower crib.....feet, and for the upper crib.....feet, measured from the top of the crib. The stringers shall

be notched 6 inches deep, but the ladder shall be set 1 inch back from the face of the crib. The timber in the ladders will be estimated and paid for as "Timber in Permanent Construction," and the iron under the classification of "Iron and Steel."

50. **Filling.**—The filling shall be of sound, hard, broken stone, in sizes ranging from ½ of a cubic foot to 5 cubic feet, but if the proportion of voids shall at any time seem unnecessarily large to the Engineer the stone shall be arranged by hand. No shaly or soft stone will be accepted. The open spaces between the face and end stringers of all the cribs shall be carefully covered as the filling progresses, beginning at the pool level, with flat stones of fairly uniform size, placed on edge and more than covering the opening. The top surfaces of all cribs shall be paved with stones not less than 10 inches deep and from 1 to 2 square feet of surface, laid with fairly close joints, and with tops flush with the top of the cribs, and with a reasonably smooth surface, all to the satisfaction of the Engineer.

Note.—If the lock-gates are to be of wood the following specifications for them may be included.

Lock-gates.—The lock-gates shall be in number and dimensions as shown on the drawings, of.....timber of the same quality as that specified in paragraph 46. All the timbers must be dressed on all sides and out of wind, and of the full size marked on the drawings after being framed. They shall receive one coat of pure linseed oil on all sides and ends as soon as dressed.

The pieces must be accurately framed and assembled in the yard first, where all ironwork must be carefully fitted on. The toes of the gates shall be left rough, and be sawed and dressed down to exact length after the gates have been finally assembled in place. This final assembling must secure a close fit at all points of quoins, toes, and sills, and proper freedom of movement.

After they have been hung and put in proper working order the gates shall receive two coats of the best red lead and oil on the timbers, and two coats of asphaltum varnish on the ironwork.

The cost of all material and labor on the gates, including the painting, shall be covered by the contractor's prices for "Iron and Steel" and for "Timber in Permanent Construction."

51. **Measurement.**—The timber for the sills and lock-gates will be measured as the net cross-section of the finished stick multiplied by its extreme length in place; the timber for the cribs and sheet-piling will be measured in place, no deduction being made for the ladder recesses or for the parts necessarily cut out in making lap-joints. Drift-bolts, raft-fastenings, nails and spikes, which form part of the permanent work, will, unless otherwise noted, be paid for at the contractor's price for "Iron and Steel." Riprap filling will be measured by taking the inside dimensions of each pen comprising the crib.

Payment for the timber in the gates will not be made until the gates have been

tested and accepted. Payment for timber will be per M. ft. B. M.; for drift-bolts per pound; and for riprap per cubic yard, all measured as hereinbefore stated.

IRON AND STEEL.*

52. **Extent.** — All iron and steel shall be furnished and set by the contractor, and shall include all fittings for the lock-gates, such as bonnets, anchorages, straps, etc.; all the gate and valve operating machinery; the valves and fittings; the ladders in the lock walls; miter-sill and other bolts, and all other iron or steel shown or referred to on the drawings or required as part of the permanent work. Iron and steel in the temporary work will not be estimated, but must be furnished by the contractor at his own expense. The contractor must supply whatever drawings may be needed in addition to those supplied by the United States.

53. **Setting Ironwork.**—All iron or steel work shall be set as shown on the drawings, and all bolts shall be set with neat Portland cement. If not set as the work progresses, drilling the holes and setting the bolts must be done by the contractor at his own expense. The contractor will be required to do, without charge, any necessary cutting or facing in placing all iron and steel.

54. **Patterns.**—All patterns must be provided by the contractor, and shall become the property of the United States. The price named for "Iron and Steel" must include the expense of making the patterns, and of their delivery, in first-class condition, and before the completion of the contract, at..............

55. **Material.**—All structural steel shall be of the grade known as medium steel, except forged work, which shall be of soft steel, and all tests and inspections shall be made at place of manufacture prior to shipment.

The tensile strength, limit of elasticity and ductility shall be determined from a standard test-piece cut from the finished material. On tests cut from material other than plates the test-piece may be planed or turned parallel throughout its entire length. The elongation shall be measured on an original length of 8 inches, except when the thickness of the finished material is $\frac{5}{16}$ inch or less, in which case the elongation shall be measured in a length equal to sixteen times the thickness; and except in rounds of $\frac{5}{8}$ inch or less in diameter, in which case the elongation shall be measured in length equal to eight times the diameter of section tested. Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

Every finished piece of steel shall be stamped with the blow or melt number. Rivet steel and small pieces may be shipped in bundles securely wired together, with the blow or melt number on a metal tag attached.

---

*Adapted from the Standard Specifications of Iron and Steel Manufacturers.

Finished bars must be free from injurious seams, flaws, or cracks, and have a workmanlike finish. The properties must be as follows:

Maximum amount of phosphorus, 0.10 per cent, for both soft and medium steel.

Soft steel, ultimate strength, 52,000 lbs. to 62,000 lbs. per square inch; elastic limit not less than one-half the ultimate strength; elongation, 25 per cent; bending test, 180 degrees flat on itself when cold, without fracture on outside of bent portion.

Medium steel, ultimate strength, 60,000 lbs. to 70,000 lbs. per square inch; elastic limit not less than one-half the ultimate strength; elongation, 22 per cent; bending test 180 degrees to a diameter equal to thickness of piece tested when cold, without fracture on outside of bent portion.

If the United States provides no mill inspector, two tests of each melt shall be made by the manufacturer, prior to shipment, one for tension and one for bending, and a certified report of the results shall be forwarded to the Engineer Officer.

The variation in cross-section or weight of more than $2\frac{1}{2}$ per cent from that specified will be sufficient cause for rejection, except in the case of sheared plates, not ordered to gauge, which may vary 5 per cent.

All castings shall be tough gray iron, free from injurious cold-shuts or blow-holes, true to pattern, well cleaned, and of a workmanlike finish. Sample pieces, 1 inch square, cast from the same heat of metal in sand molds, shall be capable of sustaining on a clear span of 4 feet 8 inches, a central load of 500 pounds when tested in the rough bar.

56. Inspection.—Inspection will be made by the authorized agent of the Engineer Officer before shipment, and before painting, and any facilities desired by him for such inspection must be furnished by the contractor without extra charge. The decision of the inspector regarding material and workmanship shall be final, and any piece rejected must be satisfactorily replaced at once by the contractor, without expense to the United States.

57. Workmanship.—All workmanship must be first-class, and especial care must be taken to secure a neat finish to the work. The rivet-holes for splice-plates of abutting members shall be so accurately spaced that when the members are brought into position the holes shall be truly opposite before the rivets are driven. The pitch of rivets in all classes of work shall not exceed 6 inches, unless shown on the drawings, nor 16 times the thinnest outside plate, nor be less than three diameters of the rivet. The rivets used shall generally be $\frac{5}{8}$, $\frac{3}{4}$, and $\frac{7}{8}$ inch diameter. The distance between the edge of any piece and the center of a rivet-hole must never be less than $1\frac{1}{4}$ inches, except for bars less than $2\frac{1}{2}$ inches wide. When practicable it shall be at least two diameters of the rivet. Rivets must completely fill the holes, have full heads concentric with the rivet, of a height not less than .6 the diameter of the rivet, and in full contact with the surface, or be countersunk, when so required, and be machine-driven wherever practicable. The diameter of the punch shall not exceed by more than $\frac{1}{16}$ inch the diameter of the rivets to be used, and all holes must be clean cut without torn or ragged

edges. Rivet-holes must be accurately spaced; the use of drift-pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to disturb the metal about the holes.

Built members must, when finished, be true and free from twists, kinks, buckles, or open joints between the component pieces. Pin-holes must be accurately bored at right angles to the axis of the piece, and where a pin passes through two or more plates, the holes must be truly opposite after fitting up.

In all cases where a steel piece in which the full strength is required has been partially heated the whole piece must be subsequently annealed. All bends in steel must be made cold, or if the degree of curvature is so great as to require heating, the whole piece must be subsequently annealed. Upset ends must be forged out of the solid bar, and not welded on. All pieces which work or connect with other pieces must be accurately assembled before shipment. Threads and other parts liable to injury during transportation must be properly protected.

58. **Alterations.**— If, during the progress of the work, it is found advisable by the United States to make any minor alterations, these must be made by the contractor without charge to the United States. If any alterations be deemed advisable which materially increase or diminish the cost of the work, the price for such alterations must be agreed upon in writing and approved by proper authority before the change is made, or nothing in addition to the contract price will be allowed.

Note.—If the lock gates are to be of steel, the following specifications for them may be used, clauses as to material, inspection, etc., being covered by the preceding paragraphs:

Lock-gates.—There will be two pairs of steel lock-gates, built as shown on the drawings.

The contractor will be required to make whatever shop drawings are required, and as soon as these are made shall furnish, free of charge, two blue-print copies of each to the Engineer.

The gates shall be riveted up and shipped in such sections as will reduce the number of field-joints to a minimum. Each gate must be fitted together complete before shipment, and all imperfections of workmanship remedied.

The framing and riveting of the gates must be done to the satisfaction of the Engineer, and must be such that each pair, when closed, will make a water-tight barrier across the lock, allowing no leak through the metal work, or between the woodwork and the metal, or between the woodwork and its bearings. The gates will be tested at the normal level of both pools as soon as practicable after the completion of the dam, and any leaks or other defects must be made good by the contractor at his own expense.

The timber cushions and fenders of the lock-gates must be of white oak, sound, clear and free from all defects, and accurately dressed to the dimensions shown. They must receive two coats of linseed oil as soon as dressed, and the first coat must be thoroughly dry before the second is applied.

After the gates have been erected complete, all the metal and timber parts shall receive two coats of..........paint, to be applied by the contractor, the cost to be included in the general price for the gates.

59. **Painting, etc.**—After inspection and before shipment all iron and steel shall be scraped free from rust or scale, and shall then receive two coats of pure red lead and oil. The parts must not be loaded for shipment until the paint is dry. All surfaces in contact with each other shall be painted with red lead before joining; all finished surfaces and screw threads are to be well coated with tallow and white lead, and all threads must be protected where liable to injury during shipment.

The contractor must provide at his own expense scales and labor for weighing the various portions of the iron and steel.

All iron and steel will be paid for at the contractor's price per pound for "Iron and Steel."

CONDUCT AND SUPERVISION OF WORK.

60. **Engineer.** — Wherever the word "Engineer" is used in these specifications it refers to the United States Engineer Officer in charge of the work, or to his authorized agents acting under his directions.

61. **Order and Inspection of Work.**—The Engineer shall have power to prescribe the order and manner of executing the work in all its parts, and work not ordered by him will not be paid for. The work will be laid out and inspected by the local Engineer, or his authorized assistants, and he and such assistants shall have power to reject materials and work which in their judgment do not conform to the specifications and drawings. Any materials so rejected shall at once be removed by the contractor from the United States property, and any work rejected shall be at once taken out and satisfactorily replaced, and no estimate or payment will be made until such materials or work shall have been so removed. In all cases of dispute upon matters relating to the work, the local Engineer shall have power to overrule the decisions of his inspectors, but the decision of the United States Engineer Officer will be accepted as final and without appeal.

62. **Stakes and Templets.**—The contractor must conform and keep to the lines and levels for the work given by the Engineer, and all stakes, benches, etc., after being placed must be protected and kept in position by the contractor, or, if necessary to the progress of the work, must be moved to new positions under the direction of the Engineer, without cost to the United States. The contractor shall furnish, at his own expense, the stakes required to lay out the work, as well as the labor required to set them. He will also be required to furnish to the satisfaction of the Engineer, and without cost to the United States, all templets, scaffolds, and platforms that may be required in cutting, setting, or laying out any part of the work, and all needed facilities and assistance, including skiffs or boats, labor, tools, appliances, and materials of all kinds, except

engineering assistance and instruments, to enable the Engineer to make any inspection or measurement connected with the work. Templates for special surfaces, such as the hollow quoins, must be cut out of zinc or galvanized iron, to be provided and shaped by the contractor.

63. **Employees.**—The contractor will be required to employ a sufficient number of capable and efficient employees who have had experience in the class of construction which they are to superintend. Any person employed by the contractor who, in the opinion of the Engineer, is incompetent, disobedient, disorderly, or otherwise unacceptable, shall at once be discharged by the contractor, at the request of the Engineer, and shall not again be employed on the work.

64. **Machinery.**—The contractor will be required to employ a sufficient number of appliances suitable for carrying on properly all portions of the work.

65. **Use of U. S. Ground.**—The contractor will have the privilege of using, during the progress of the work, subject to the approval of the Engineer, any portions of the land to which the United States has title on both sides of the river at the site (for stone-yards, temporary buildings, etc.), which are not used or reserved by the United States. It is understood, however, that the contractor shall, at any time during the progress of the work, promptly vacate and clean up any part of the Government ground that may have been allotted to, or been used by him, whenever such part is needed for any purpose by the United States. The contractor shall keep the grounds and all the buildings within the United States limit, which are occupied or used by himself or his employees, in a cleanly and thoroughly sanitary condition. He will not be allowed to rent or assign to other parties any buildings erected on the United States' land without the permission of the Engineer.

66. **Work on Sundays, etc.**—In cases of extraordinary emergency, to be determined by the Engineer, work on Sundays or legal holidays may be required. With this exception no Sunday work will be permitted, except for repairs to plant and work of similar nature, nor will any night work be permitted which requires the presence of an inspector.

67. **Failure to Prosecute or to Protect Work.**—The contractor shall use such methods and appliances for the performance of all the operations connected with the work embraced in these specifications as will secure a satisfactory quality of work and such rate of progress as will, in the opinion of the Engineer, insure the completion of the work within the contract time. If at any time the contractor shall refuse or fail to prosecute the work, or to provide for carrying on the same as directed by the Engineer, or fail to protect properly any part of the work, permanent or temporary, the Engineer shall have power to employ men, to purchase or otherwise provide materials, tools, machinery, etc., and to put the work in proper advancement or condition, and the excess of cost of doing so shall be deducted from payments to be made under this contract.

The failure of the contractor to do a fair amount of work in any one month, or to make satisfactory progress for the same period of time, as determined by the Engineer,

may be taken as sufficient cause for annulment of this contract, as provided for in the form of contract to be entered into or for action under the provisions of these specifications, as the best interests of the United States may demand.

68. **Annulment of Contract.**—In case of annulment of this contract the United States shall have the right to retain all materials, tools, buildings, tramways, cars, etc., or any part or parts of same prepared for or in use in the prosecution of the work, together with any or all leases, rights of way or quarry privileges, under purchase, at a valuation to be determined by the Engineer.

69. **Removal of Rubbish.**—Within thirty days after completion of the work, and before the final payment is made, the contractor shall remove from the site all rubbish, old and unused material, piles, cribs, etc., and shall fill up all excavations made for his convenience, as the Engineer shall direct, and shall leave the whole site thoroughly clean and in good order and condition, without expense to the United States.

70. **Special Labor to be Supplied.**—The contractor shall furnish, when required by the Engineer, at the fixed prices of......cents per man per hour for unskilled labor, and of......cents per man per hour for skilled labor, all the labor necessary for carrying out such special work as may be required by the Engineer, and which is not covered in other clauses of this contract.

71. **Complete Work Required.**—The contractor is not to take advantage of any omission of details in drawings or specifications, or errors in either, but he will be required to do everything which may be necessary to carry out in good faith this contract, which contemplates complete structures, in good working order, of good material, and accurate workmanship, skillfully fitted and properly connected and put together. Any point not clearly understood is to be referred to the Engineer for decision.

72. **Changes.**—Should any changes in the details of the work, or any of its parts, including any changes of the shape, arrangement, or fitting of the parts, be deemed necessary or advisable, and be ordered by the Engineer before these details or parts have been finished, the changes must be made by the contractor at the same unit prices as those of the bid; but anything which materially increases the cost of the work is not to be done until first ordered in writing by the Engineer Officer.

73. **Damage to Vessels.**—The contractor shall be responsible for all damages or injury caused by the permanent or temporary works to any vessels, steamboats, tows, barges, etc., during the continuance of his contract. In no case will the United States defend any suit brought for such damages inflicted before the works have been finally accepted by the United States. Any injury to the works by such vessels, etc., while the works are in the hands of the contractor, and before delivery to the United States, shall be repaired by the contractor to the satisfaction of the Engineer, without expense to the United States.

74. **Lighting Work.**—From sunset to sunrise the work shall be properly lighted to prevent accident to boats navigating the river. The lighting shall be done by the

contractor at his own expense, and he shall be liable for any and all damage due to neglect in this particular.

QUANTITIES.

75. From the nature of the work it is impossible to estimate with accuracy the quantities of material required. The amounts named below will be used in canvassing the bids; but the bidders must so fix their prices as to permit increase or diminution in the amounts required, within the limits stated for each; and it is understood and agreed that such increase or diminution, whether resulting from error in estimate or from modification of plans, shall form no basis for any claim against the United States. The quantities given do not take into account nails, spikes, bolts, tie-rods, etc., to be furnished by the contractor and included in his price for other materials in the temporary work:

Classification.	Unit.	Approximate Quantities.	May be Increased. (Per cent.)	May be Decreased. (Per cent.)
Bolt holes in rock or masonry	Lin. ft.		50	30
Coffer-dam filling	Cu. yds.		50	20
Coffer-dam timber	M. ft. B. M.		20	20
Concrete	Cu. yds		20	20
Deposit	Cu. yds.		200	100
Drift-bolts	Pounds		20	20
Embankment	Cu. yds.		50	30
Excavation, earth	Cu. yds.		40	20
Excavation, rock	Cu. yds.		50	50
Iron and steel	Pounds		20	20
Puddling	Cu. yds.		30	30
Paving	Sq. yds.		20	20
Piles	Lin. ft.		30	30
Riprap	Cu. yds.		50	30
Timber in permanent construction	M. ft. B. M.		20	20

(If the lock is to be of stone, the amounts of the various classes of masonry should be given.)

76. **Measurement.**—Measurement for payment of all work and material shall be made net, in place, unless otherwise specified.

77. **Losses.**—Material of any kind lost or damaged through the fault or negligence of the contractor shall be replaced by him at his own expense. Material of any kind lost or damaged by natural causes before that portion of the work in which it is placed has been accepted, shall be replaced by the contractor at his own expense.

78. **Materials not Mentioned.**—Payment will be made to the contractor only as hereinbefore specified. All materials and work not mentioned as to be estimated for payment, and which may be necessary to complete the work, shall be furnished by the contractor at his own expense.

SPECIFICATIONS FOR AMERICAN PORTLAND CEMENT.*

1. The cement shall be an American Portland, dry and free from lumps. By a Portland cement is meant the product obtained from the heating or calcining up to incipient fusion of intimate mixtures, either natural or artificial, of argillaceous with calcareous substances, the calcined product to contain at least 1.7 times as much of lime, by weight, as of the materials which give the lime its hydraulic properties, and to be finely pulverized after said calcination, and thereafter additions or substitutions for the purpose only of regulating certain properties of technical importance to be allowable to not exceeding 2 per cent of the calcined product.

2. The cement shall be put up in strong, sound barrels well lined with paper, so as to be reasonably protected against moisture, or in stout cloth or canvas sacks. Each package shall be plainly labeled with the name of the brand and of the manufacturer. Any package broken or containing damaged cement may be rejected or accepted as a fractional package, at the option of the United States agent in local charge.

3. Bidders will state the brand of cement which they propose to furnish. The right is reserved to reject a tender for any brand which has not established itself as a high-grade Portland cement and has not for three years or more given satisfaction in use under climatic or other conditions of exposure of at least equal severity to those of the work proposed.

4. Tenders will be received only from manufacturers or their authorized agents.

(The following paragraph will be substituted for paragraphs 3 and 4 above, when cement is to be furnished and placed by the contractor:

No cement will be allowed to be used except established brands of high-grade Portland cement which have been made by the same mill and in successful use under similar climatic conditions to those of the proposed work for at least three years.)

5. The average weight per barrel shall not be less than 375 pounds net. Four sacks shall contain one barrel of cement. If the weight, as determined by test weighings, is found to be below 375 pounds per barrel, the cement may be rejected, or, at the option of the Engineer Officer in charge, the contractor may be required to supply, free of cost to the United States, an additional amount of cement equal to the shortage.

6. Tests may be made of the fineness, specific gravity, soundness, time of setting, and tensile strength of the cement.

7. **Fineness.**—Ninety-two per cent of the cement must pass through a sieve made of No. 40 wire, Stubb's gauge, having 10,000 openings per square inch.

8. **Specific Gravity.**—The specific gravity of the cement, as determined from a sample which has been carefully dried, shall be between 3.10 and 3.25.

---

* These specifications for Portland, Natural, and Puzzolan cements are the standards of the Engineer Department, U. S. Army, and are reprinted from the Report of the Board of Engineer Officers on testing Hydraulic Cements, June, 1901. (Professional Papers, Corps of Engineers, U. S. Army, No. 28.)

9. **Soundness.**—To test the soundness of the cement, at least two pats of neat cement mixed for five minutes with 20 per cent of water by weight shall be made on glass, each pat about 3 inches in diameter and $\frac{1}{2}$ inch thick at the center, tapering thence to a thin edge. The pats are to be kept under a wet cloth until finally set, when one is to be placed in fresh water for twenty-eight days. The second pat will be placed in water which will be raised to the boiling-point for six hours, then allowed to cool. Neither should show distortion or cracks. The boiling test may or may not reject, at the option of the Engineer Officer in charge.

10. **Time of Setting.**—The cement shall not acquire its initial set in less than forty-five minutes and must have acquired its final set in ten hours.

(The following paragraph will be substituted for the above in case a quick-setting cement is desired:

The cement shall not acquire its initial set in less than twenty nor more than thirty minutes, and must have acquired its final set in not less than forty-five minutes nor in more than two and one-half hours.)

The pats made to test the soundness may be used in determining the time of setting. The cement is considered to have acquired its initial set when the pat will bear, without being appreciably indented, a wire $\frac{1}{12}$ inch in diameter loaded to weigh $\frac{1}{4}$ pound. The final set has been acquired when the pat will bear, without being appreciably indented, a wire $\frac{1}{24}$ inch in diameter loaded to weigh 1 pound.

11. **Tensile Strength.**—Briquettes made of neat cement, after being kept in air for twenty-four hours under a wet cloth, and the balance of the time in water, shall develop tensile strength per square inch as follows:

After seven days, 450 pounds; after twenty-eight days, 540 pounds.

Briquettes made of 1 part cement and 3 parts standard sand, by weight, shall develop tensile strength per square inch as follows:

After seven days, 140 pounds; after twenty-eight days, 220 pounds.

(In case quick-setting cement is desired, the following tensile strengths will be substituted for the above:

Neat briquettes: After seven days, 400 pounds; after twenty-eight days, 480 pounds.

Briquettes of 1 part cement to 3 parts standard sand: After seven days, 120 pounds; after twenty-eight days, 180 pounds.)

12. The highest result from each set of briquettes made at any one time is to be considered the governing test. Any cement not showing an increase of strength in the twenty-eight-day tests over the seven-day tests will be rejected.

13. When making briquettes neat cement will be mixed with 20 per cent of water by weight, and sand and cement with $12\frac{1}{2}$ per cent of water by weight. After being thoroughly mixed and worked for five minutes, the cement or mortar will be placed in the briquette mold in four equal layers, and each layer rammed and compressed by thirty blows of a soft brass or copper rammer $\frac{3}{4}$ of an inch in diameter (or $\frac{7}{16}$ of an inch

square, with rounded corners), weighing 1 pound. It is to be allowed to drop on the mixture from a height of about ½ inch. When the ramming has been completed, the surplus cement shall be struck off and the final layer smoothed with a trowel held almost horizontal and drawn back with sufficient pressure to make its edge follow the surface of the mold.

14. The above are to be considered the minimum requirements. Unless a cement has been recently used on work under this office, bidders will deliver a sample barrel for test before the opening of bids. If this sample shows higher tests than those given above, the average of tests made on subsequent shipments must come up to those found with the sample.

15. A cement may be rejected in case it fails to meet any of the above requirements. An agent of the contractor may be present at the making of the tests, or, in case of the failure of any of them, they may be repeated in his presence. If the contractor so desires, the Engineer Officer in charge may, if he deems it to the interest of the United States, have any or all of the tests made or repeated at some recognized standard testing laboratory in the manner herein specified. All expenses of such tests shall be paid by the contractor, and all such tests shall be made on samples furnished by the Engineer Officer from cement actually delivered to him.

SPECIFICATIONS FOR NATURAL CEMENT.

1. The cement shall be a freshly packed Natural or Rosendale, dry, and free from lumps. By Natural cement is meant one made by calcining natural rock at a heat below incipient fusion, and grinding the product to powder.

2. The cement shall be put up in strong, sound barrels, well lined with paper so as to be reasonably protected against moisture, or in stout cloth or canvas sacks. Each package shall be plainly labeled with the name of the brand and of the manufacturer. Any package broken or containing damaged cement may be rejected, or accepted as a fractional package, at the option of the United States agent in local charge.

3. Bidders will state the brand of cement which they propose to furnish. The right is reserved to reject a tender for any brand which has not given satisfaction in use under climatic or other conditions of exposure of at least equal severity to those of the work proposed.

4. Tenders will be received only from manufacturers or their authorized agents.

(The following paragraph will be substituted for paragraphs 3 and 4 above when cement is to be furnished and placed by the contractor:

No cement will be allowed to be used except established brands of high-grade natural cement which have been in successful use under similar climatic conditions to those of the proposed work.)

5. The average net weight per barrel shall not be less than 300 pounds. (West of

the Allegheny Mountains this may be 265 pounds.) Three sacks of cement shall have the same weight as 1 barrel. If the average net weight, as determined by test weighings, is found to be below 300 pounds (265 pounds) per barrel, the cement may be rejected, or, at the option of the Engineer Officer in charge, the contractor may be required to supply, free of cost to the United States, an additional amount of cement equal to the shortage.

6. Tests may be made of the fineness, time of setting, and tensile strength of the cement.

7. **Fineness.**—At least 80 per cent of the cement must pass through a sieve made of No. 40 wire, Stubb's gauge, having 10,000 openings per square inch.

8. **Time of Setting.**—The cement shall not acquire its initial set in less than twenty minutes and must have acquired its final set in four hours.

9. The time of setting is to be determined from a pat of neat cement mixed for five minutes with 30 per cent of water by weight and kept under a wet cloth until finally set. The cement is considered to have acquired its initial set when the pat will bear, without being appreciably indented, a wire $\frac{1}{12}$ inch in diameter loaded to weigh $\frac{1}{4}$ pound. The final set has been acquired when the pat will bear, without being appreciably indented, a wire $\frac{1}{24}$ inch in diameter loaded to weigh 1 pound.

10. **Tensile Strength.**—Briquettes made of neat cement shall develop the following tensile strengths per square inch, after having been kept in air for twenty-four hours under a wet cloth and the balance of the time in water:

At the end of seven days, 90 pounds; at the end of twenty-eight days, 200 pounds.

Briquettes made of one part cement and one part standard sand by weight shall develop the following tensile strengths per square inch:

After seven days, 60 pounds; after twenty-eight days, 150 pounds.

11. The highest result from each set of briquettes made at any one time is to be considered the governing test. Any cement not showing an increase of strength in the twenty-eight-day tests over the seven-day tests will be rejected.

12. The neat cement for briquettes shall be mixed with 30 per cent of water by weight, and the sand and cement with 17 per cent of water by weight. After being thoroughly mixed and worked for five minutes the cement or mortar is to be placed in the briquette mold in four equal layers, each of which is to be rammed and compressed by thirty blows of a soft brass or copper rammer $\frac{3}{4}$ of an inch in diameter (or $\frac{7}{10}$ of an inch square with rounded corners), weighing 1 pound. It is to be allowed to drop on the mixture from a height of about $\frac{1}{2}$ inch. Upon the completion of the ramming the surplus cement shall be struck off and the last layer smoothed with a trowel held nearly horizontal and drawn back with sufficient pressure to make its edge follow the surface of the mold.

13. The above are to be considered the minimum requirements. Unless a cement has been recently used on work under this office, bidders will deliver a sample barrel for test before the opening of the bids. Any cement showing by sample higher tests than those given must maintain the average so shown in subsequent deliveries.

14. A cement may be rejected which fails to meet any of the above requirements. An agent of the contractor may be present at the making of the tests, or, in case of the failure of any of them, they may be repeated in his presence. If the contractor so desires, the Engineer Officer may, if he deems it to the interest of the United States, have any or all of the tests made or repeated at some recognized standard testing laboratory in the manner above specified. All expenses of such tests shall be paid by the contractor, and all such tests shall be made on samples furnished by the Engineer Officer from cement actually delivered to him.

SPECIFICATIONS FOR PUZZOLAN CEMENT.

1. The cement shall be a Puzzolan of uniform quality, finely and freshly ground, dry, and free from lumps, made by grinding together without subsequent calcination granulated blast-furnace slag with slaked lime.

2. The cement shall be put up in strong, sound barrrels, well lined with paper, so as to be reasonably protected against moisture, or in stout cloth or canvas sacks. Each package shall be plainly labeled with the name of the brand and of the manufacturer. Any package broken or containing damaged cement may be rejected, or accepted as a fractional package, at the option of the United States agent in local charge.

3. Bidders will state the brand of cement which they propose to furnish. The right is reserved to reject a tender for any brand which has not given satisfaction in use under climatic or other conditions of exposure of at least equal severity to those of the work proposed, and for any brand from cement works that do not make and test the slag used in the cement.

4. Tenders will be received only from manufacturers or their authorized agents.

(The following paragraph will be substituted for paragraphs 3 and 4 above when cement is to be furnished and placed by the contractor:

No cement will be allowed to be used except established brands of high-grade Puzzolan cement which have been in successful use under similar climatic conditions to those of the proposed work, and which come from cement works that make the slag used in the cement.)

5. The average weight per barrel shall not be less than 330 pounds net. Four sacks shall contain 1 barrel of cement. If the weight as determined by test weighings is found to be below 330 pounds per barrel, the cement may be rejected, or, at the option of the Engineer Officer in charge, the contractor may be required to supply, free of cost to the United States, an additional amount of cement equal to the shortage.

6. Tests may be made of the fineness, specific gravity, soundness, time of setting, and tensile strength of the cement.

7. **Fineness.**—Ninety-seven per cent of the cement must pass through a sieve made of No. 40 wire, Stubb's gauge, having 10,000 openings per square inch.

8. Specific Gravity.—The specific gravity of the cement, as determined from a sample which has been carefully dried, shall be between 2.7 and 2.8.

9. Soundness.—To test the soundness of cement, pats of neat cement mixed for five minutes with 18 per cent of water by weight shall be made on glass, each pat about 3 inches in diameter and $\frac{1}{2}$ inch thick at the center, tapering thence to a thin edge. The pats are to be kept under wet cloths until finally set, when they are to be placed in fresh water. They should not show distortion or cracks at the end of twenty-eight days.

10. Time of Setting.—The cement shall not acquire its initial set in less than forty-five minutes and shall acquire its final set in ten hours. The pats made to test the soundness may be used in determining the time of setting. The cement is considered to have acquired its initial set when the pat will bear, without being appreciably indented, a wire $\frac{1}{12}$ inch in diameter loaded to $\frac{1}{4}$ pound weight. The final set has been acquired when the pat will bear, without being appreciably indented, a wire $\frac{1}{24}$ inch in diameter loaded to 1 pound weight.

11. Tensile Strength.—Briquettes made of neat cement, after being kept in air under a wet cloth for twenty-four hours and the balance of the time in water, shall develop tensile strengths per square inch as follows:

After seven days, 350 pounds; after twenty-eight days, 500 pounds.

Briquettes made of one part cement and three parts standard sand by weight shall develop tensile strength per square inch as follows:

After seven days, 140 pounds; after twenty-eight days, 220 pounds.

12. The highest result from each set of briquettes made at any one time is to be considered the governing test. Any cement not showing an increase of strength in the twenty-eight-day tests over the seven-day tests will be rejected.

13. When making briquettes neat cement will be mixed with 18 per cent of water by weight, and sand and cement with 10 per cent of water by weight. After being thoroughly mixed and worked for five minutes the cement or mortar will be placed in the briquette mold in four equal layers, and each layer rammed and compressed by thirty blows of a soft brass or copper rammer, $\frac{3}{4}$ of an inch in diameter or $\frac{7}{10}$ of an inch square, with rounded corners, weighing 1 pound. It is to be allowed to drop on the mixture from a height of about $\frac{1}{2}$ inch. When the ramming has been completed the surplus cement shall be struck off and the final layer smoothed with a trowel held almost horizontal and drawn back with sufficient pressure to make its edge follow the surface of the mold.

14. The above are to be considered the minimum requirements. Unless a cement has been recently used on work under this office, bidders will deliver a sample barrel for test before the opening of bids. If this sample shows higher tests than those given above, the average of tests made on subsequent shipments must come up to those found with the sample.

15. A cement may be rejected in case it fails to meet any of the above requirements. An agent of the contractor may be present at the making of the tests, or, in

case of the failure of any of them, they may be repeated in his presence. If the contractor so desires, the Engineer Officer in charge may, if he deems it to the interest of the United States, have any or all of the tests made or repeated at some recognized testing laboratory in the manner herein specified, all expenses of such tests to be paid by the contractor. All such tests shall be made on samples furnished by the Engineer Officer from cement actually delivered to him.

APPENDIX C.

LAWS FOR THE PROTECTION AND PRESERVATION OF THE NAVIGABLE WATERS OF THE UNITED STATES.

EXTRACT FROM THE RIVER AND HARBOR ACT APPROVED MARCH 3, 1899.

Stats. L., vol. 30, pp.

Construction of bridges—consent of Congress necessary for.

SEC. 9. That it shall not be lawful to construct or commence the construction of any bridge, dam, dike, or causeway over or in any port, roadstead, haven, harbor, canal, navigable river, or other navigable water of the United States until the consent of Congress to the building of such structures shall have been obtained and until the plans for the same shall have been submitted to and approved by the Chief of Engineers and by the Secretary of War:

Bridges may be built under State legislation.

Provided, That such structures may be built under authority of the legislature of a State across rivers and other waterways the navigable portions of which lie wholly within the limits of a single State, provided the location and plans thereof are submitted to and approved by the Chief of Engineers and by the Secretary of War before construction is commenced:

Plans must be approved before construction is begun.

And provided further, That when plans for any bridge or other structure have been approved by the Chief of Engineers and by the Secretary of War it shall not be lawful to deviate from such plans either before or after completion of the structure unless the modification of said plans has previously been submitted to and received the approval of the Chief of Engineers and of the Secretary of War.

Approved plans must be adhered to.

Creation of obstructions forbidden.

SEC. 10. That the creation of any obstruction not affirmatively authorized by Congress, to the navigable capacity of any of the waters of the United States is hereby prohibited; and it shall not be lawful to build or commence the building of any wharf, pier, dolphin, boom, weir, breakwater, bulkhead, jetty, or other structures in any port, roadstead, haven, harbor, canal, navigable river, or other water of the United States, outside established harbor lines, or where no harbor lines have been established, except on plans recommended by the Chief of Engineers and authorized by the Secretary of War;

Construction of wharves, etc.

Alteration, etc., of channels.

and it shall not be lawful to excavate or fill, or in any manner to alter or modify the course, location, condition, or capacity of, any port, roadstead, haven, harbor, canal, lake, harbor of refuge, or inclosure within the limits of any

breakwater, or of the channel of any navigable water of the United States, unless the work has been recommended by the Chief of Engineers and authorized by the Secretary of War prior to beginning the same.

Harbor lines, establishment of.

SEC. 11. That where it is made manifest to the Secretary of War that the establishment of harbor lines is essential to the preservation and protection of harbors he may, and is hereby authorized to cause such lines to be established, beyond which no piers, wharves, bulkheads, or other works shall be extended or deposits made, except under such regulations as may be prescribed from time to time by him: *Provided*, That whenever the Secretary of War grants to any person or persons permission to extend piers, wharves, bulkheads, or other works, or to make deposits in any tidal harbor or river of the United States beyond any harbor lines established under authority of the United States, he shall cause to be ascertained the amount of tide water displaced by any such structure or by any such deposits, and he shall, if he deem it necessary, require the parties to whom the permission is given to make compensation for such displacement either by excavating in some part of the harbor, including tide-water channels between high and low water mark, to such an extent as to create a basin for as much tide water as may be displaced by such structure or by such deposits, or in any other mode that may be satisfactory to him.

Compensation for tide water displaced by structures and deposits.

Penalties for violations of three preceding sections.

SEC. 12. That every person and every corporation that shall violate any of the provisions of sections nine, ten, and eleven of this Act, or any rule or regulation made by the Secretary of War in pursuance of the provisions of the said section fourteen, shall be deemed guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding twenty-five hundred dollars nor less than five hundred dollars, or by imprisonment (in the case of a natural person) not exceeding one year, or by both such punishments, in the discretion of the court. And further, the removal of any structures or parts of structures erected in violation of the provisions of the said sections may be enforced by the injunction of any circuit court exercising jurisdiction in any district in which such structures may exist, and proper proceedings to this end may be instituted under the direction of the Attorney-General of the United States.

Removal of unlawful structures.

Deposits of refuse, etc., forbidden.

SEC. 13. That it shall not be lawful to throw, discharge, or deposit, or cause, suffer, or procure to be thrown, discharged, or deposited either from or out of any ship, barge, or other floating craft of any kind, or from the shore, wharf, manufacturing establishment, or mill of any kind, any refuse matter of any kind or description whatever other than that flowing from streets and sewers and passing therefrom in a liquid state, into any navigable water of the United States, or into any tributary of any navigable water from which the same shall float or be washed into such navigable water; and it shall not

be lawful to deposit, or cause, suffer, or procure to be deposited material of any kind in any place on the bank of any navigable water, or on the bank of any tributary of any navigable water, where the same shall be liable to be washed into such navigable water, either by ordinary or high tides, or by storms or floods, or otherwise, whereby navigation shall or may be impeded or obstructed: *Provided*, That nothing herein contained shall extend to, apply to, or prohibit the operations in connection with the improvement of navigable waters or construction of public works, considered necessary and proper by the United States officers supervising such improvement or public work: *And provided further*, That the Secretary of War, whenever in the judgment of the Chief of Engineers anchorage and navigation will not be injured thereby, may permit the deposit of any material above mentioned in navigable waters, within limits to be defined and under conditions to be prescribed by him, provided application is made to him prior to depositing such material; and whenever any permit is so granted the conditions thereof shall be strictly complied with, and any violation thereof shall be unlawful.

Lawful deposits.

Deposits by permits.

Injuries to Government works, etc., in navigable waters.

SEC. 14. That it shall not be lawful for any person or persons to take possession of or make use of for any purpose, or build upon, alter, deface, destroy, move, injure, obstruct by fastening vessels thereto or otherwise, or in any manner whatever impair the usefulness of any sea-wall, bulkhead, jetty, dike, levee, wharf, pier, or other work built by the United States, or any piece of plant, floating or otherwise, used in the construction of such work under the control of the United States, in whole or in part, for the preservation and improvement of any of its navigable waters or to prevent floods, or as boundary marks, tide gauges, surveying stations, buoys, or other established marks, nor remove for ballast or other purposes any stone or other material composing such works: *Provided*, That the Secretary of War may, on the recommendation of the Chief of Engineers, grant permission for the temporary occupation or use of any of the aforementioned public works when in his judgment such occupation or use will not be injurious to the public interest.

Permits for occupation of public works.

Anchoring or sinking vessels in navigable channels forbidden.

SEC. 15. That it shall not be lawful to tie up or anchor vessels or other craft in navigable channels in such a manner as to prevent or obstruct the passage of other vessels or craft; or to voluntarily or carelessly sink, or permit or cause to be sunk, vessels or other craft in navigable channels; or to float loose timber and logs, or to float what is known as sack rafts of timber and logs in streams or channels actually navigated by steamboats in such manner as to obstruct, impede, or endanger navigation. And whenever a vessel, raft, or other craft is wrecked and sunk in a navigable channel, accidentally or otherwise, it shall be the duty of the owner of such sunken craft to immediately mark it with a buoy or beacon during the day and a lighted

Sunken vessels to be marked.

lantern at night, and to maintain such marks until the sunken craft is removed or abandoned, and the neglect or failure of the said owner so to do shall be unlawful; and it shall be the duty of the owner of such sunken craft to commence the immediate removal of the same, and prosecute such removal diligently, and failure to do so shall be considered as an abandonment of such craft, and subject the same to removal by the United States as hereinafter provided for.

Failure to remove sunken vessels unlawful.

SEC. 16. That every person and every corporation that shall violate, or that shall knowingly aid, abet, authorize, or instigate a violation of the provisions of sections thirteen, fourteen, and fifteen of this Act shall be guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding twenty-five hundred dollars nor less than five hundred dollars, or by imprisonment (in the case of a natural person) for not less than thirty days nor more than one year, or by both such fine and imprisonment, in the discretion of the court, one-half of said fine to be paid to the person or persons giving information which shall lead to conviction. And any and every master, pilot, and engineer, or person or persons acting in such capacity, respectively, on board of any boat or vessel who shall knowingly engage in towing any scow, boat, or vessel loaded with any material specified in section thirteen of this Act to any point or place of deposit or discharge in any harbor or navigable water, elsewhere than within the limits defined and permitted by the Secretary of War, or who shall willfully injure or destroy any work of the United States contemplated in section fourteen of this Act, or who shall willfully obstruct the channel of any waterway in the manner contemplated in section fifteen of this Act, shall be deemed guilty of a violation of this Act, and shall upon conviction be punished as hereinbefore provided in this section, and shall also have his license revoked or suspended foi a term to be fixed by the judge before whom tried and convicted. And any boat, vessel, scow, raft, or other craft used or employed in violating any of the provisions of sections thirteen, fourteen, and fifteen of this Act shall be liable for the pecuniary penalties specified in this section, and in addition thereto for the amount of the damages done by said boat, vessel, scow, raft, or other craft, which latter sum shall be placed to the credit of the appropriation for the improvement of the harbor or waterway in which the damage occurred, and said boat, vessel, scow, raft, or other craft may be proceeded against summarily by way of libel in any district court of the United States having jurisdiction thereof.

Penalties for violations of sections 13, 14, 15.

Liability of masters, pilots, etc.

Libel against boats violating prohibitions.

SEC. 17. That the Department of Justice shall conduct the legal proceedings necessary to enforce the foregoing provisions of sections nine to sixteen, inclusive, of this Act; and it shall be the duty of district attorneys of the United States to vigorously prosecute all offenders against the same

Department of Justice to enforce the law.

United States attorneys to prosecute offenders.

whenever requested to do so by the Secretary of War or by any of the officials hereinafter designated, and it shall furthermore be the duty of said district attorneys to report to the Attorney-General of the United States the action taken by them against offenders so reported, and a transcript of such reports shall be transmitted to the Secretary of War by the Attorney-General; and for the better enforcement of the said provisions and to facilitate the detection and bringing to punishment of such offenders, the officers and agents of the United States in charge of river and harbor improvements, and the assistant engineers and inspectors employed under them by authority of the Secretary of War, and the United States collectors of customs and other revenue officers, shall have power and authority to swear out process and to arrest and take into custody, with or without process, any person or persons who may commit any of the acts or offenses prohibited by the aforesaid sections of this Act, or who may violate any of the provisions of the same: *Provided*, That no person shall be arrested without process for any offense not committed in the presence of some one of the aforesaid officials: *And provided further*, That whenever any arrest is made under the provisions of this Act, the person so arrested shall be brought forthwith before a commissioner, judge, or court of the United States for examination of the offenses alleged against him; and such commissioner, judge, or court shall proceed in respect thereto as authorized by law in case of crimes against the United States.

Officers and employees of United States to arrest offenders.

Parties arrested to be given a hearing.

Bridges obstructing navigation.

SEC. 18. That whenever the Secretary of War shall have good reason to believe that any railroad or other bridge now constructed, or which may hereafter be constructed, over any of the navigable waterways of the United States is an unreasonable obstruction to the free navigation of such waters on account of insufficient height, width of span, or otherwise, or where there is difficulty in passing the draw opening or the draw span of such bridge by rafts, steamboats, or other water craft, it shall be the duty of the said Secretary, first giving the parties reasonable opportunity to be heard, to give notice to the persons or corporations owning or controlling such bridge so to alter the same as to render navigation through or under it reasonably free, easy, and unobstructed; and in giving such notice he shall specify the changes recommended by the Chief of Engineers that are required to be made, and shall prescribe in each case a reasonable time in which to make them. If at the end of such time the alteration has not been made, the Secretary of War shall forthwith notify the United States district attorney for the district in which such bridge is situated, to the end that the criminal proceedings hereinafter mentioned may be taken. If the persons, corporation, or association owning or controlling any railroad or other bridge shall, after receiving notice to that effect, as hereinbefore required, from the Sec-

Notice of alterations.

Proceedings in case of default in making alterations.

Penalty for default in making alterations.

retary of War, and within the time prescribed by him willfully fail or refuse to remove the same or to comply with the lawful order of the Secretary of War in the premises, such persons, corporation, or association shall be deemed guilty of a misdemeanor, and on conviction thereof shall be punished by a fine not exceeding five thousand dollars, and every month such persons, corporation, or association shall remain in default in respect to the removal or alteration of such bridge shall be deemed a new offense, and subject the persons, corporation, or association so offending to the penalties above prescribed: *Provided*, That in any case arising under the provisions of this section an appeal or writ of error may be taken from the district courts or from the existing circuit courts direct to the Supreme Court either by the United States or by the defendants.

Appeal.

Removal of wrecks.

SEC. 19. That whenever the navigation of any river, lake, harbor, sound, bay, canal, or other navigable waters of the United States shall be obstructed or endangered by any sunken vessel, boat, water craft, raft, or other similar obstruction, and such obstruction has existed for a longer period than thirty days, or whenever the abandonment of such obstruction can be legally established in a less space of time, the sunken vessel, boat, water craft, raft, or other obstruction shall be subject to be broken up, removed, sold, or otherwise disposed of by the Secretary of War at his discretion, without liability for any damage to the owners of the same: *Provided*, That in his discretion, the Secretary of War may cause reasonable notice of such obstruction of not less than thirty days, unless the legal abandonment of the obstruction can be established in a less time, to be given by publication, addressed "To whom it may concern," in a newspaper published nearest to the locality of the obstruction, requiring the removal thereof: *And provided also*, That the Secretary of War may, in his discretion, at or after the time of giving such notice, cause sealed proposals to be solicited by public advertisement, giving reasonable notice of not less than ten days, for the removal of such obstruction as soon as possible after the expiration of the above specified thirty days' notice, in case it has not in the meantime been so removed, these proposals and contracts, at his discretion, to be conditioned that such vessel, boat, water craft, raft, or other obstruction, and all cargo and property contained therein, shall become the property of the contractor, and the contract shall be awarded to the bidder making the proposition most advantageous to the United States: *Provided*, That such bidder shall give satisfactory security to execute the work: *Provided further*, That any money received from the sale of any such wreck, or from any contractor for the removal of wrecks, under this paragraph shall be covered into the Treasury of the United States.

May be broken up and removed without liability.

Proposals for removal may be invited.

Money received from sales of wrecks to be deposited in Treasury.

SEC. 20. That under emergency, in the case of any vessel, boat, water

In emergent cases Secretary of War may take immediate possession of and remove wrecks.

craft, or raft, or other similar obstruction, sinking or grounding, or being unnecessarily delayed in any Government canal or lock, or in any navigable waters mentioned in section nineteen, in such manner as to stop, seriously interfere with, or specially endanger navigation, in the opinion of the Secretary of War, or any agent of the United States to whom the Secretary may delegate proper authority, the Secretary of War or any such agent shall have the right to take immediate possession of such boat, vessel, or other water craft, or raft, so far as to remove or to destroy it and to clear immediately the canal, lock, or navigable waters aforesaid of the obstruction thereby caused, using his best judgment to prevent any unnecessary injury; and no one shall interfere with or prevent such removal or destruction: *Provided*, That the officer or agent charged with the removal or destruction of an obstruction under this section may in his discretion give notice in writing to the owners of any such obstruction requiring them to remove it: *And provided further*, That the expense of removing any such obstruction as aforesaid shall be a charge against such craft and cargo; and if the owners thereof fail or refuse to reimburse the United States for such expense within thirty days after notification, then the officer or agent aforesaid may sell the craft or cargo, or any part thereof that may not have been destroyed in removal, and the proceeds of such sale shall be covered into the Treasury of the United States.

Expense of removing to be a charge against vessel and cargo.

Appropriation for removing wrecks.

Such sum of money as may be necessary to execute this section and the preceding section of this Act is hereby appropriated out of any money in the Treasury not otherwise appropriated, to be paid out on the requisition of the Secretary of War.

Repeal of previous laws.

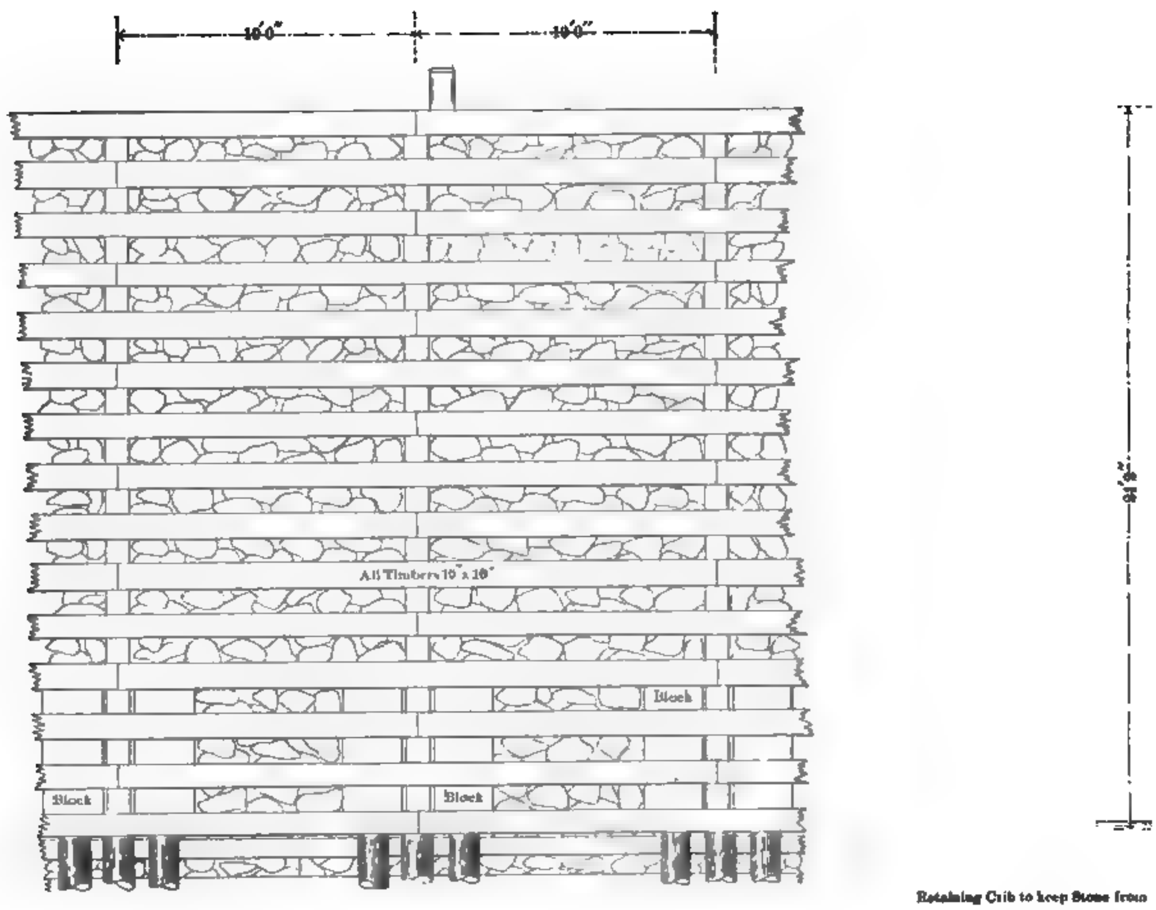
That all laws or parts of laws inconsistent with the foregoing sections ten [nine] to twenty, inclusive, of this Act are hereby repealed: *Provided*, That no action begun, or right of action accrued, prior to the passage of this Act shall be affected by this repeal.

EXTRACT FROM THE RIVER AND HARBOR ACT APPROVED JUNE 13, 1902.

SEC. 11. That section four of the River and Harbor Act of August eighteenth, eighteen hundred and ninety-four, be, and is hereby, amended so as to read as follows:

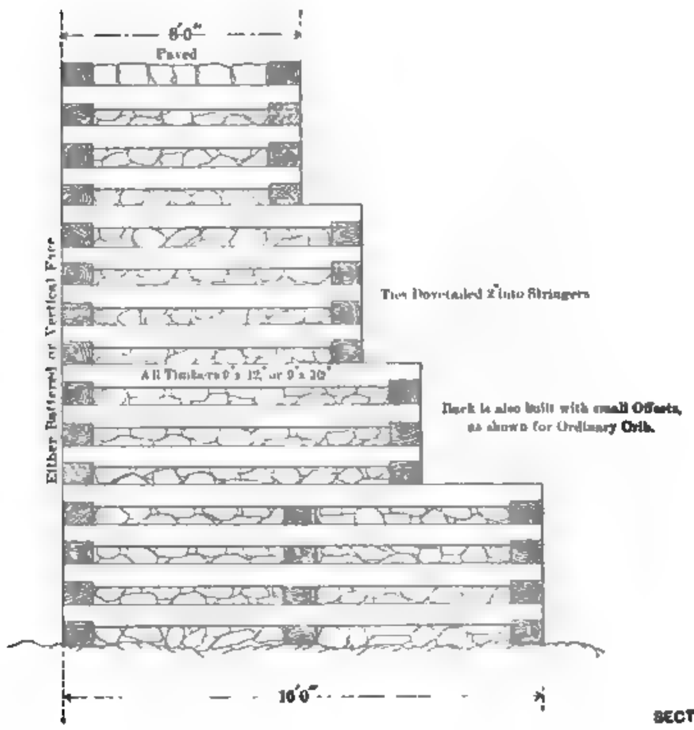
"SEC. 4. That it shall be the duty of the Secretary of War to prescribe such rules and regulations for the use, administration, and navigation of any or all canals and similar works of navigation that now are, or that hereafter may be, owned, operated, or maintained by the United States as in his judgment the public necessity may require; and he is also authorized to prescribe regulations to govern the speed and movement of vessels and other water craft in any public navigable channel which has

been improved under authority of Congress, whenever, in his judgment, such regulations are necessary to protect such improved channels from injury, or to prevent interference with the operations of the United States in improving navigable waters or injury to any plant that may be employed in such operations. Such rules and regulations shall be posted, in conspicuous and appropriate places, for the information of the public; and every person and every corporation which shall violate such rules and regulations shall be deemed guilty of a misdemeanor, and, on conviction thereof in any district court of the United States within whose territorial jurisdiction such offense may have been committed, shall be punished by a fine of not exceeding five hundred dollars, or by imprisonment (in the case of a natural person) not exceeding six months, in the discretion of the court."



Retaining Crib to keep Stone from falling out into the Entrance.

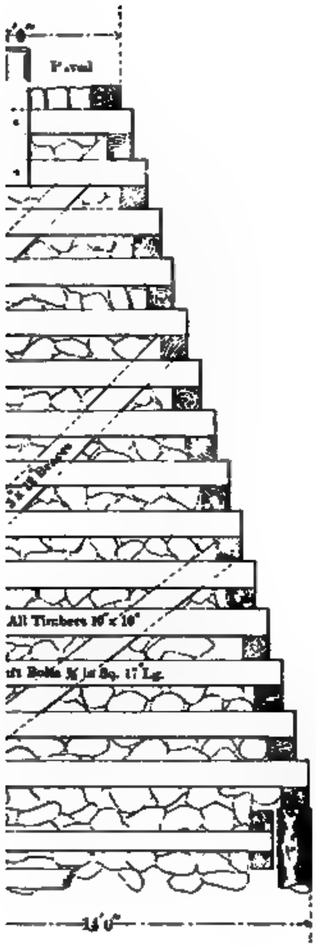
ELEVATION AND SECTION OF ORDINARY CRIB.

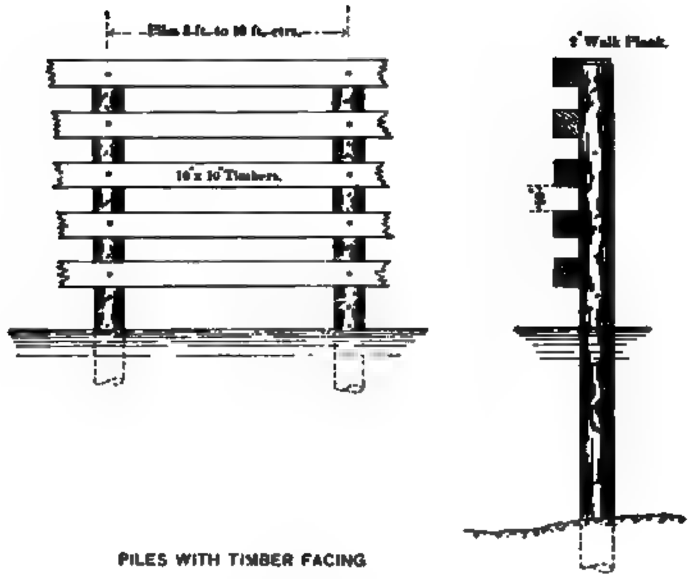


SECTION OF FRAMED CRIB.

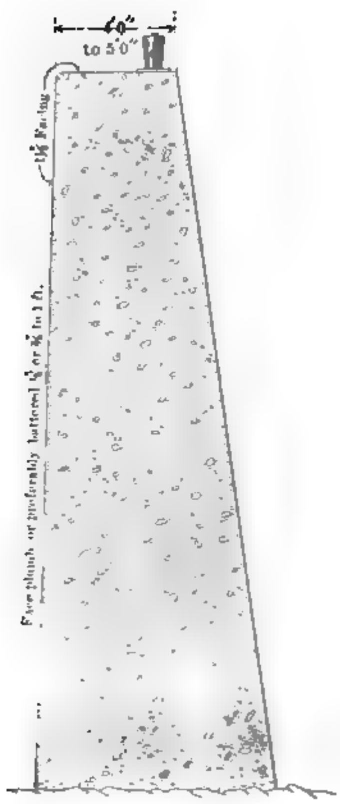
SECTION OF CONCRETE GUARD CRIB (FO

For Guide Cribs wider bases are used if there





PILES WITH TIMBER FACING

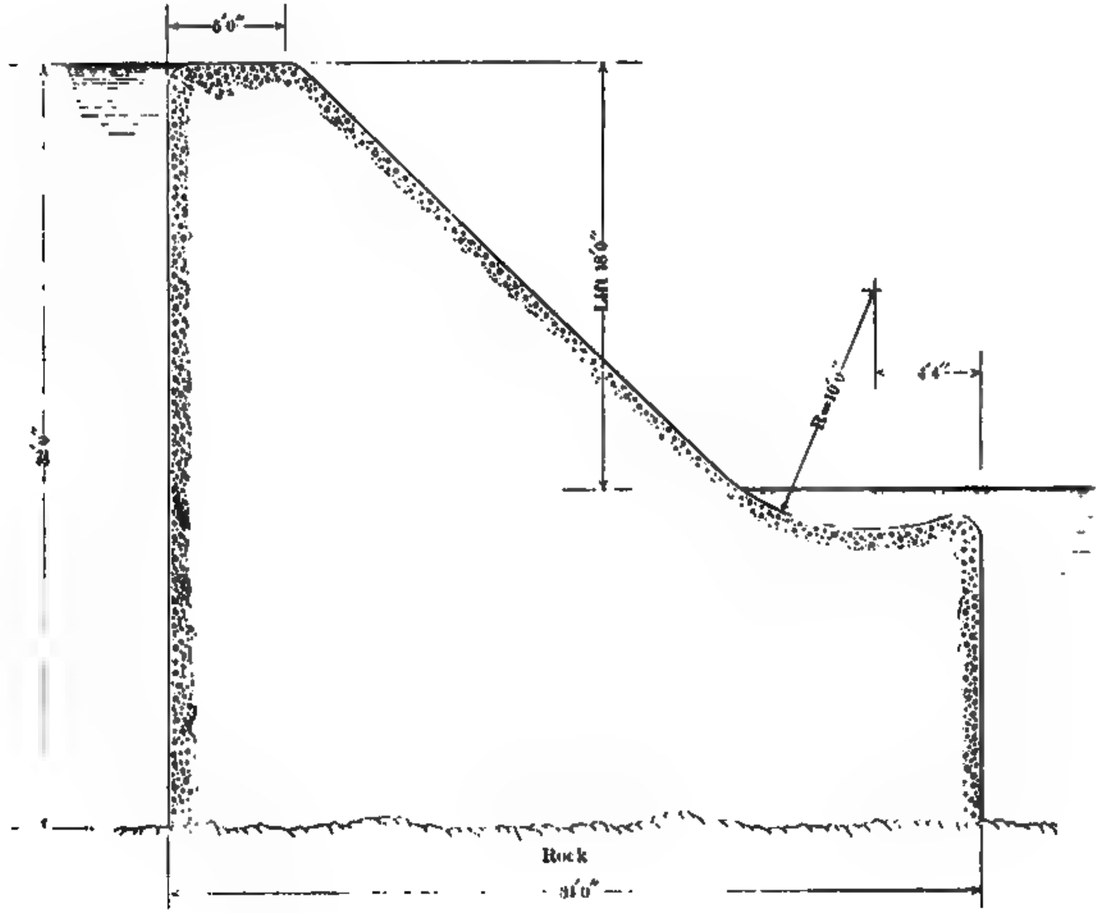


TYPES OF GUIDE CRIBS AS USED ON TRIBUTARIES OF THE OHIO RIVER.

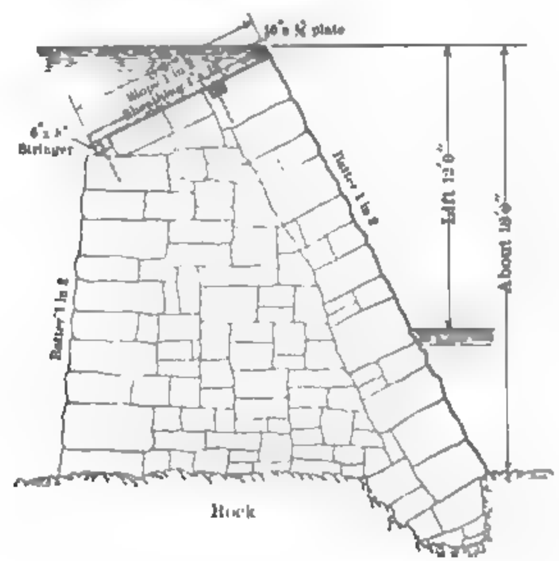
Scale:

12" 0' 1' 2' 3' 4'

NOTE.—Guide cribs are below and above land walls of locks. Guard cribs are below and above river walls of locks.



CONCRETE DAM.
No. 9, Kentucky River, Ky. 1902
Length 250 ft. Proportions about 1 of Portland Cement to 12 of sand and limestone. Facing 1 to 2, 1½ inches thick.

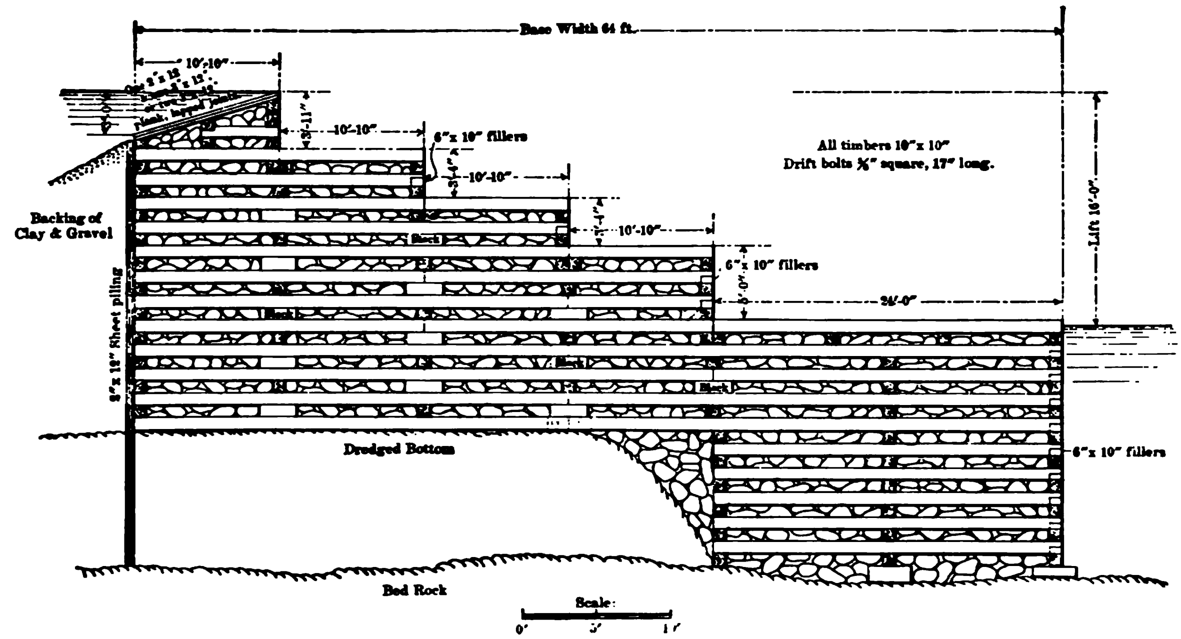


RUBBLE MASONRY DAM.
(No. 4 Black Warrior River, Alabama. 1902.)
Length 640 ft. laid in Portland Cement mortar.

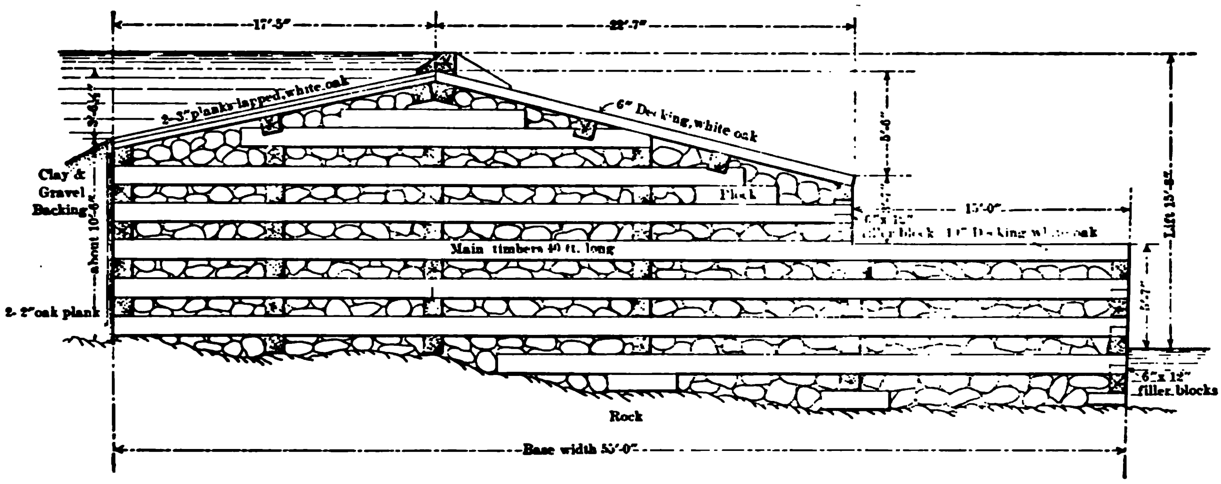
EXAMPLES OF MASONRY DAMS IN AMERICA.

Scale:

0' 2' 4' 6' 8' 10'

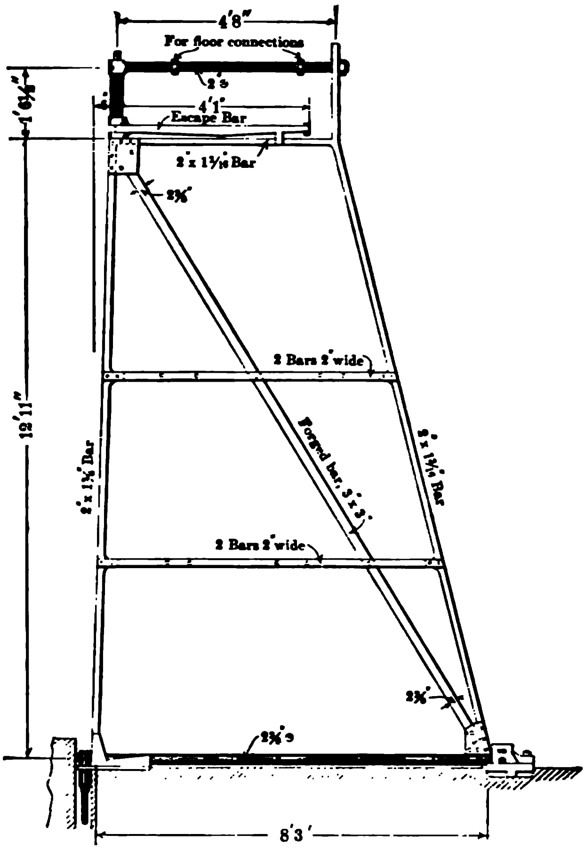


SECTION OF STEP DAM FOR RIVERS OF HIGH FLOODS, AFTER EXAMPLES BUILT IN 1896–1900.

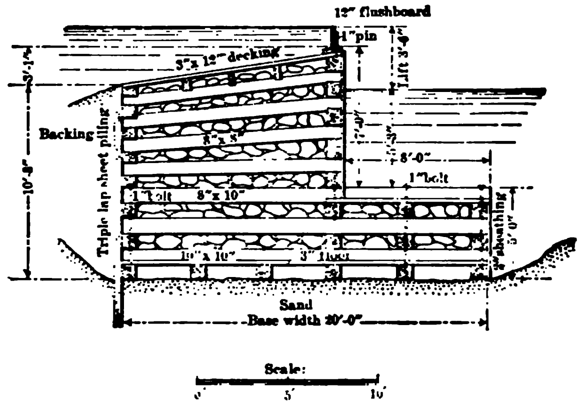


SLOPE DAM WITH COMB-STICK AND LONG UP-STREAM SLOPE.
(Dam No. 2, Green River, Ky., 1897.)

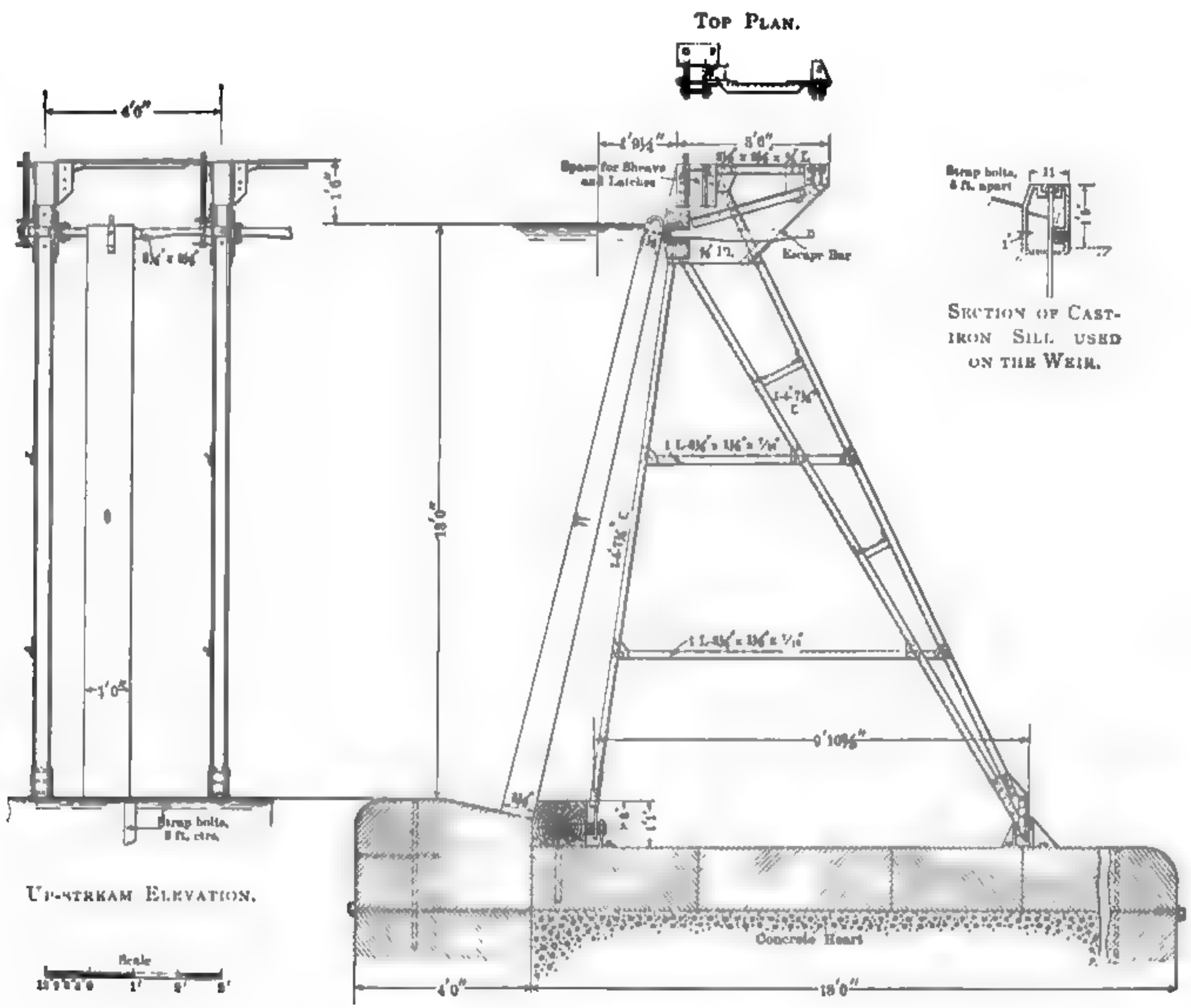
EXAMPLES OF FIXED DAMS IN AMERICA, OF TIMBER CRIBS FILLED WITH STONE.



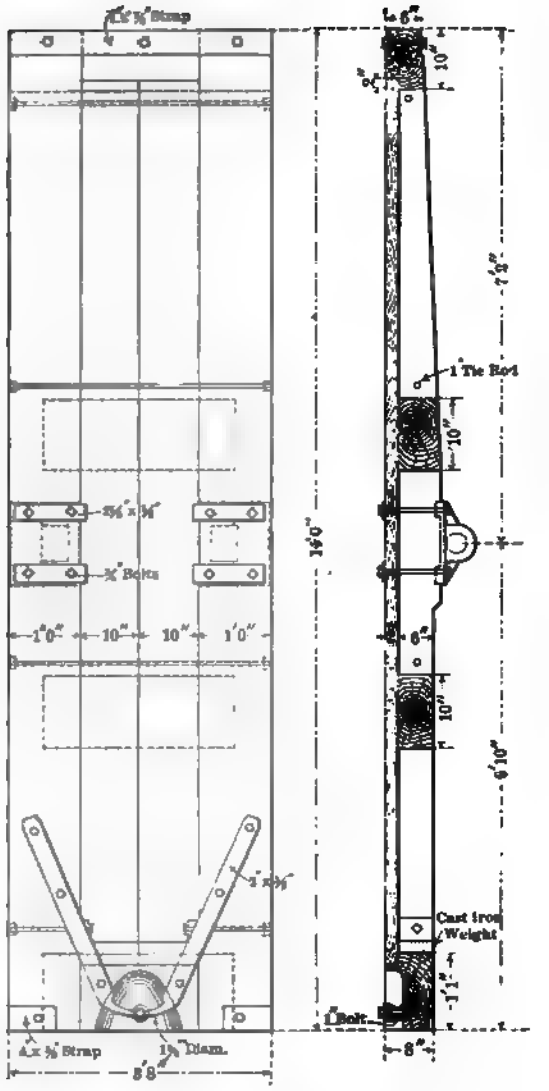
Pass Trestle of the Klecan Needle Dam on the River Moldau, Bohemia, 1899.
(Trestles are spaced 1½ meters apart.)



Section of Timber Dam as used on the Fox River, Wisconsin, 1898.
(Ordinary flood range, about 3 feet.)



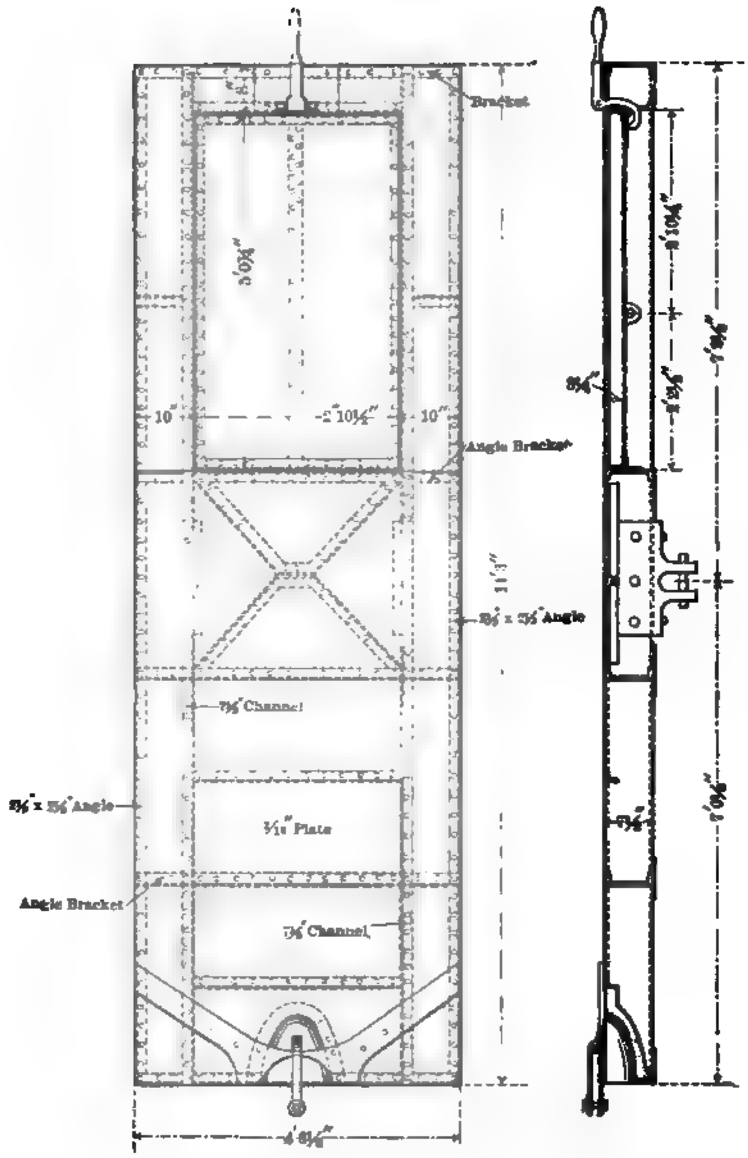
Pass Trestle and Needle, Louisa Dam, Big Sandy River, Ky., 1896.



ELEVATION OF UPSTREAM FACE. SECTION NEAR CENTER.

WOODEN WICKET—PASSES OF KANAWHA AND OHIO RIVERS, U.S.A.

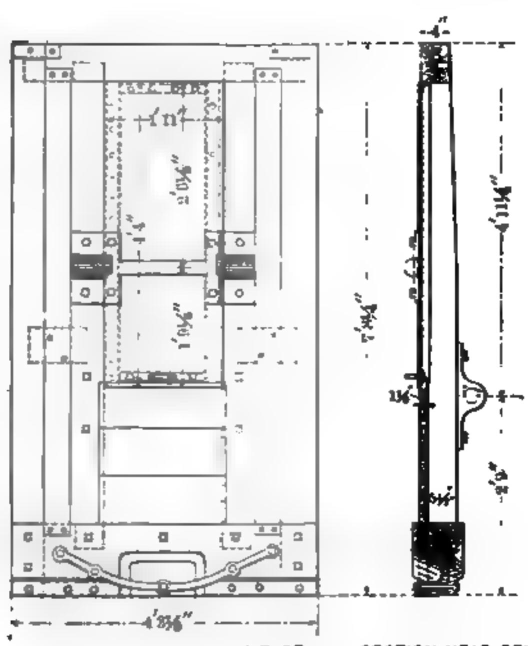
(1880-1900.)



ELEVATION OF UPSTREAM FACE. SECTION NEAR CENTER.

IRON WICKET—PASS OF LA MULATIÈRE DAM, ON THE SAÔNE, NEAR LYONS, FRANCE.

(1879.)



ELEVATION OF UPSTREAM FACE. SECTION NEAR CENTER.

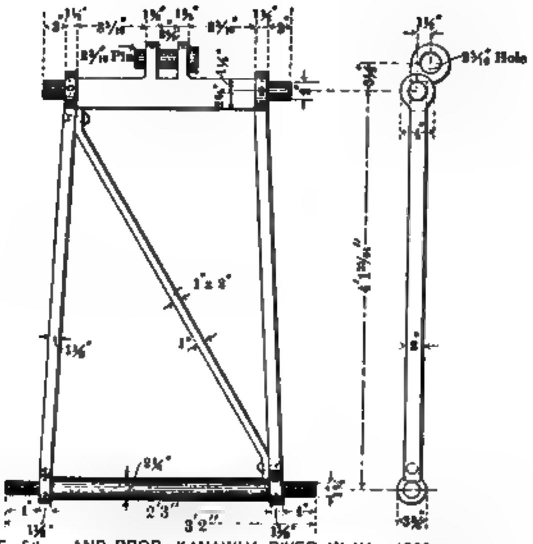
WEIR WICKET USED ON THE BELGIAN MEUSE.

(About 1876.)

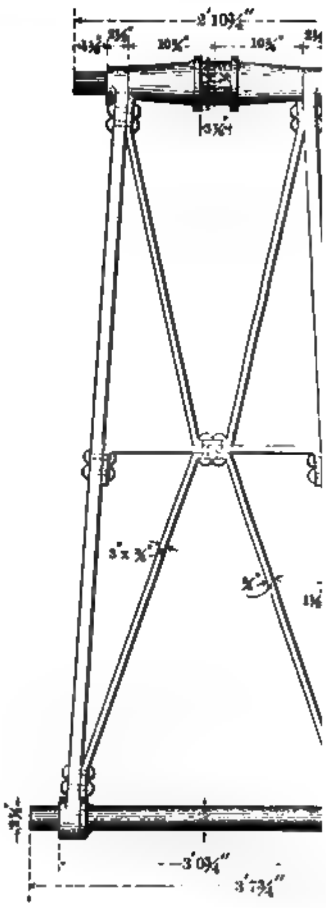
DETAILS OF CHANOINE WICKETS.

Scale of Feet

12" 6" 0" 1' 2' 3'



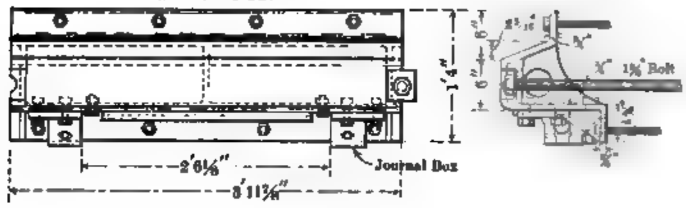
WEIR HORSE SILL, AND PROP, KANAWHA RIVER, W. VA. 1896

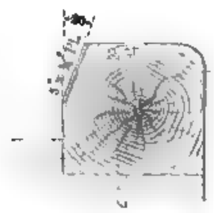


PASS HORSE, DAM No. 6, OHIO
1899

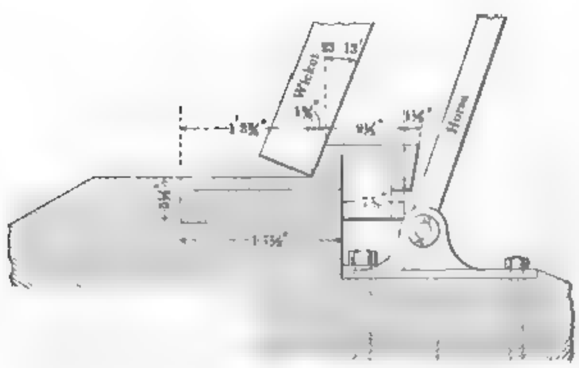
PLAN OF SILL.

END ELEVATION.





PASS PROP, DAM No. 6, OHIO RIVER

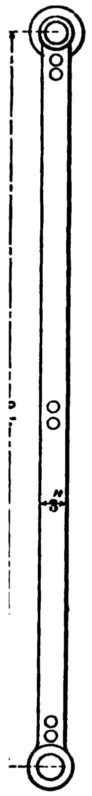


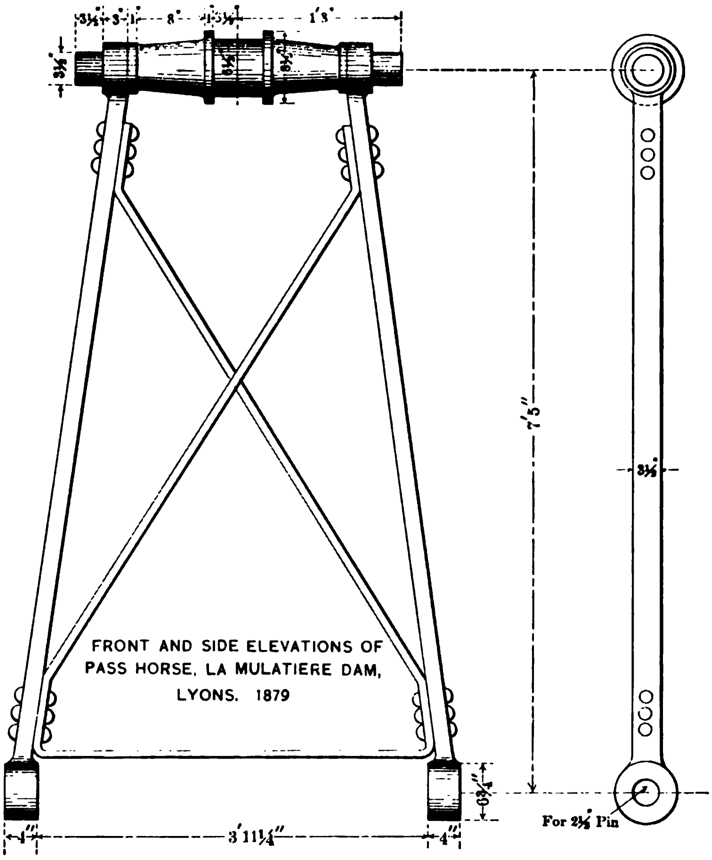
SECTION OF SILL OF LA MULATIERE DAM.

DETAILS OF HORSE
OF CHANOINE

Sc

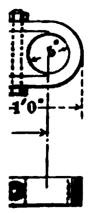
12″ 0′

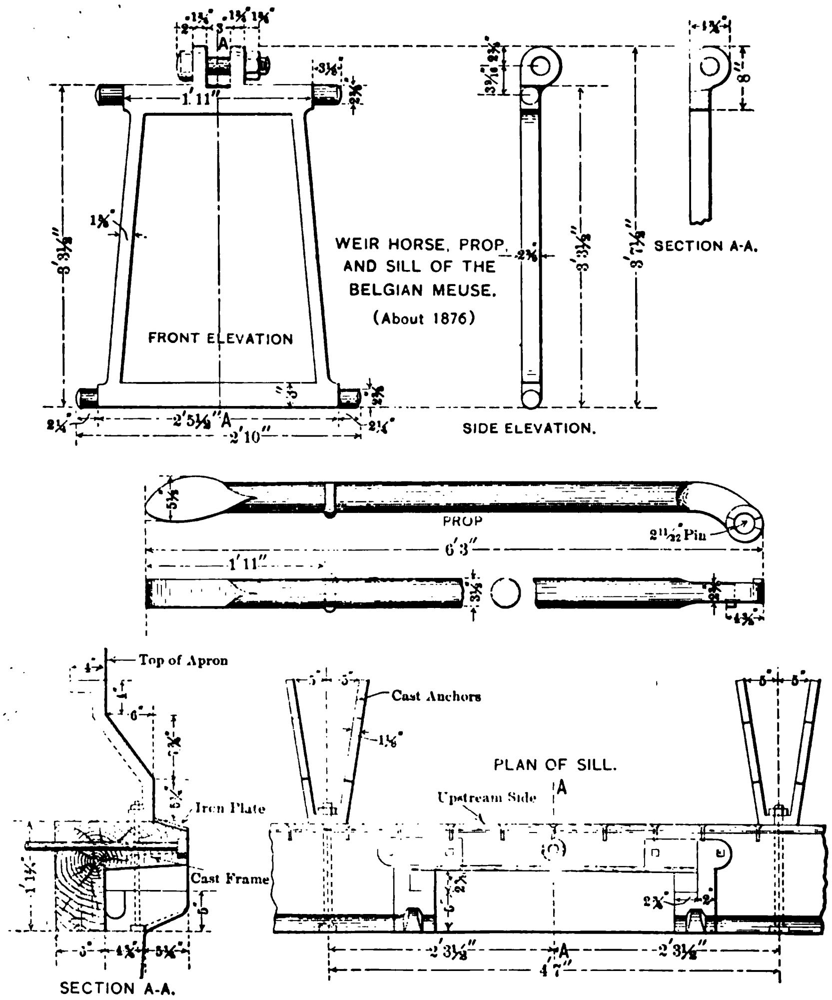




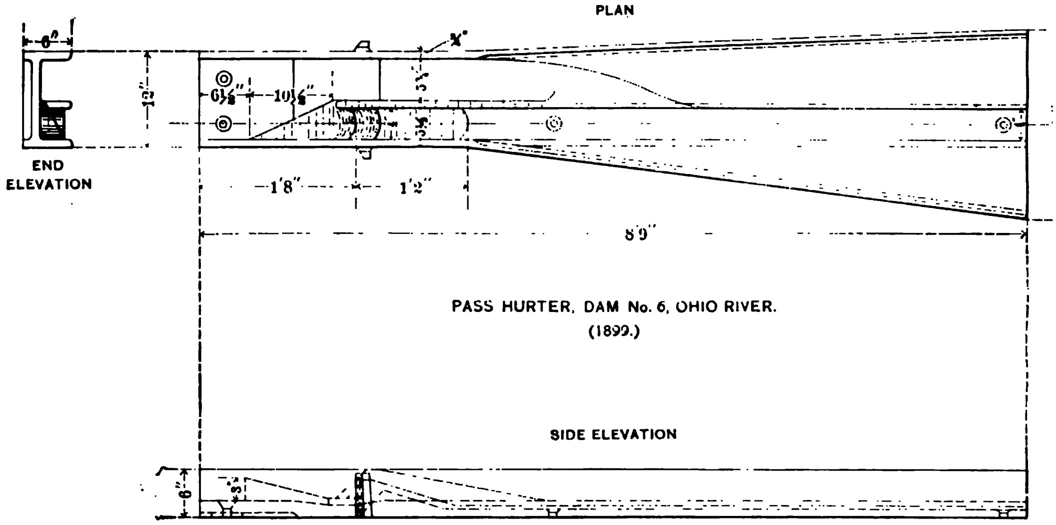
mber

ɔF PASS SILL,
ıM No. 6

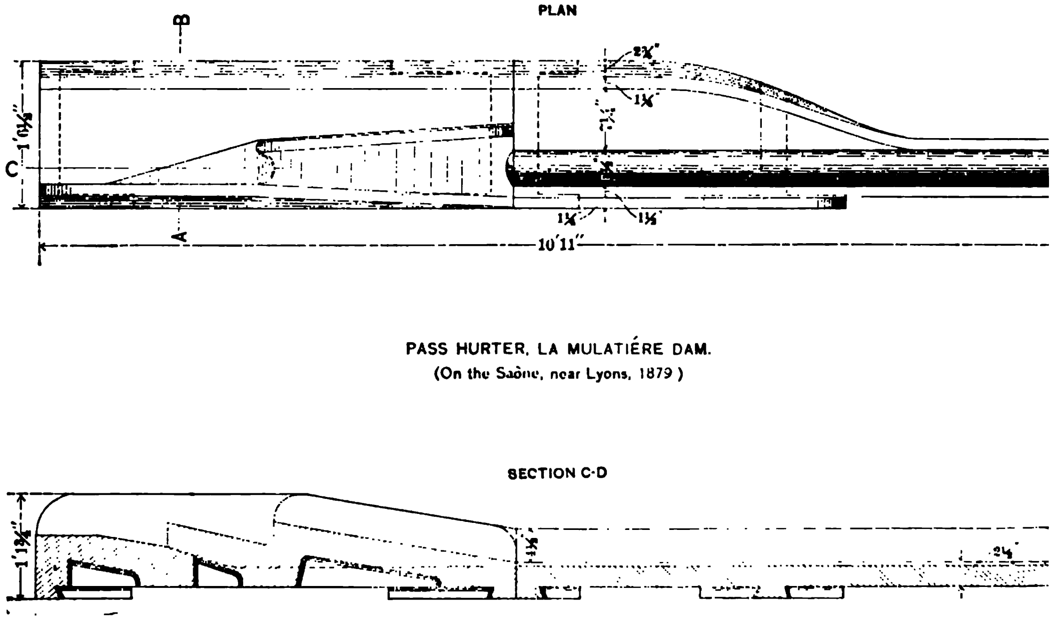




ɔS, AND SILLS
T DAMS.

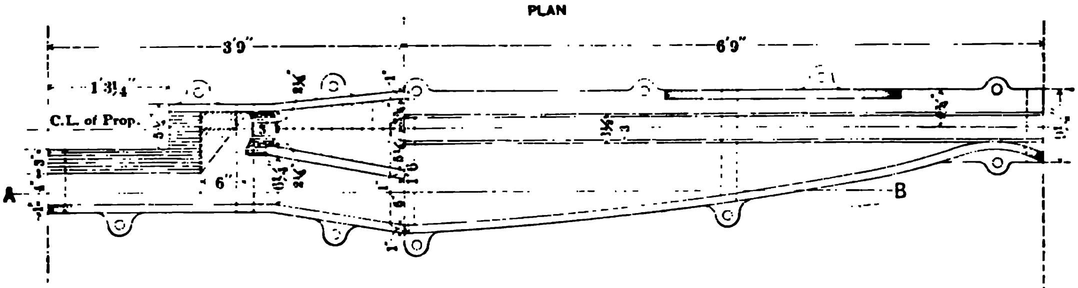


PASS HURTER, DAM No. 6, OHIO RIVER.
(1899.)

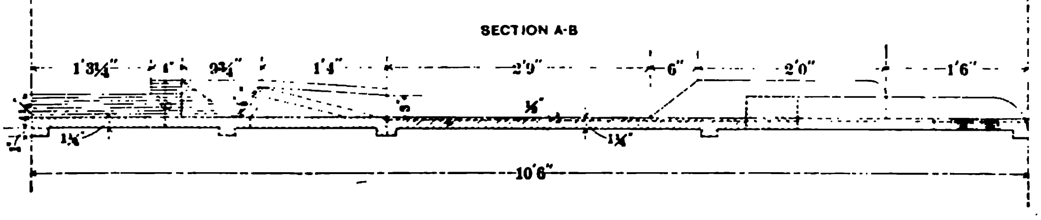


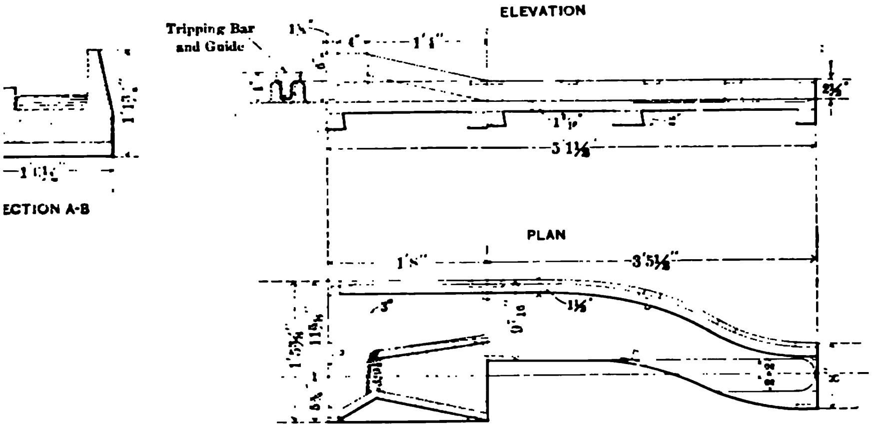
PASS HURTER, LA MULATIÉRE DAM.
(On the Saône, near Lyons, 1879.)

TYPES OF HURTERS U

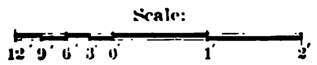


PASS HURTER. KANAWHA RIVER, W. VA.
(1896.)

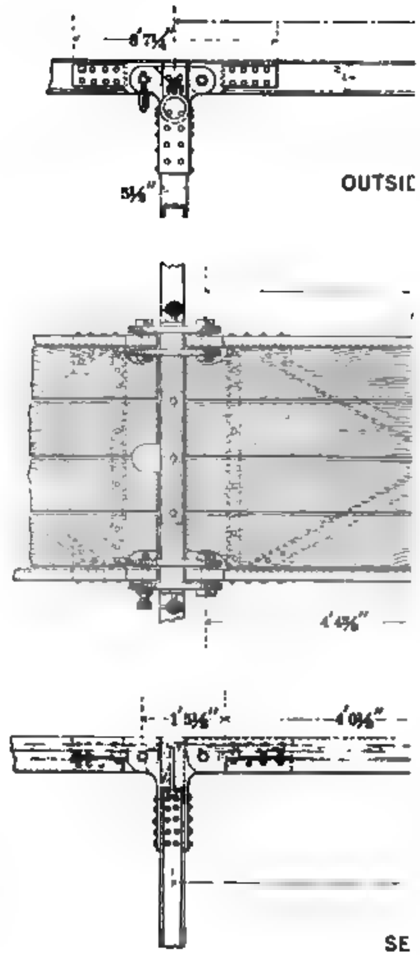




WEIR HURTER OF THE BELGIAN MEUSE.
FOR USE WITH TRIPPING BAR.
(About 1876.)

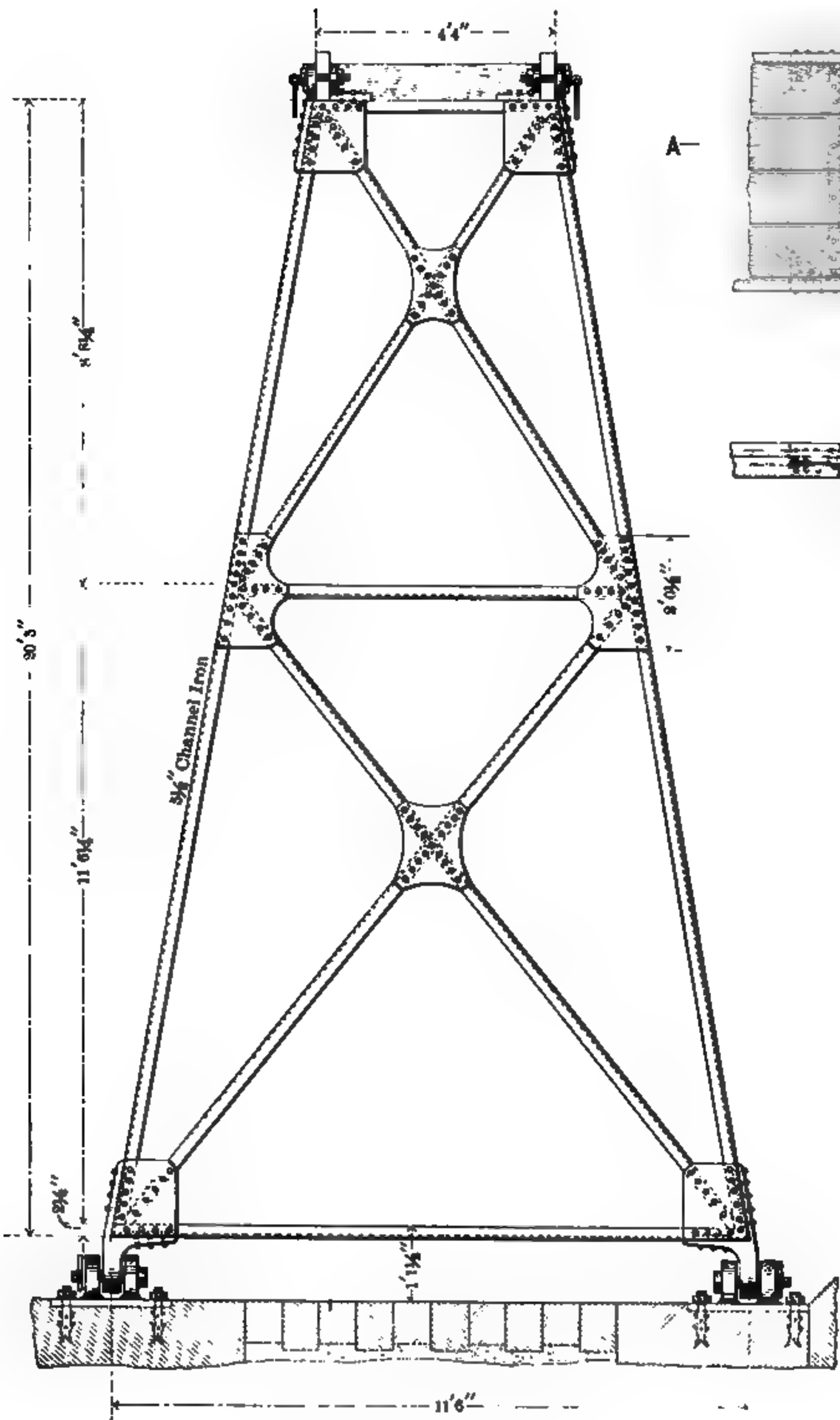


CHANOINE WICKET DAMS.

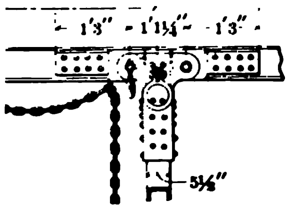


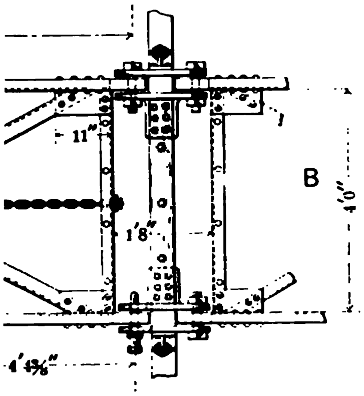
FLOORS OF LA

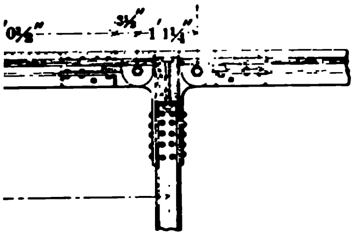
TRESTL
CH/



PASS TRESTLE, LA MULATIERE DAM.
(Saône, 1879.)



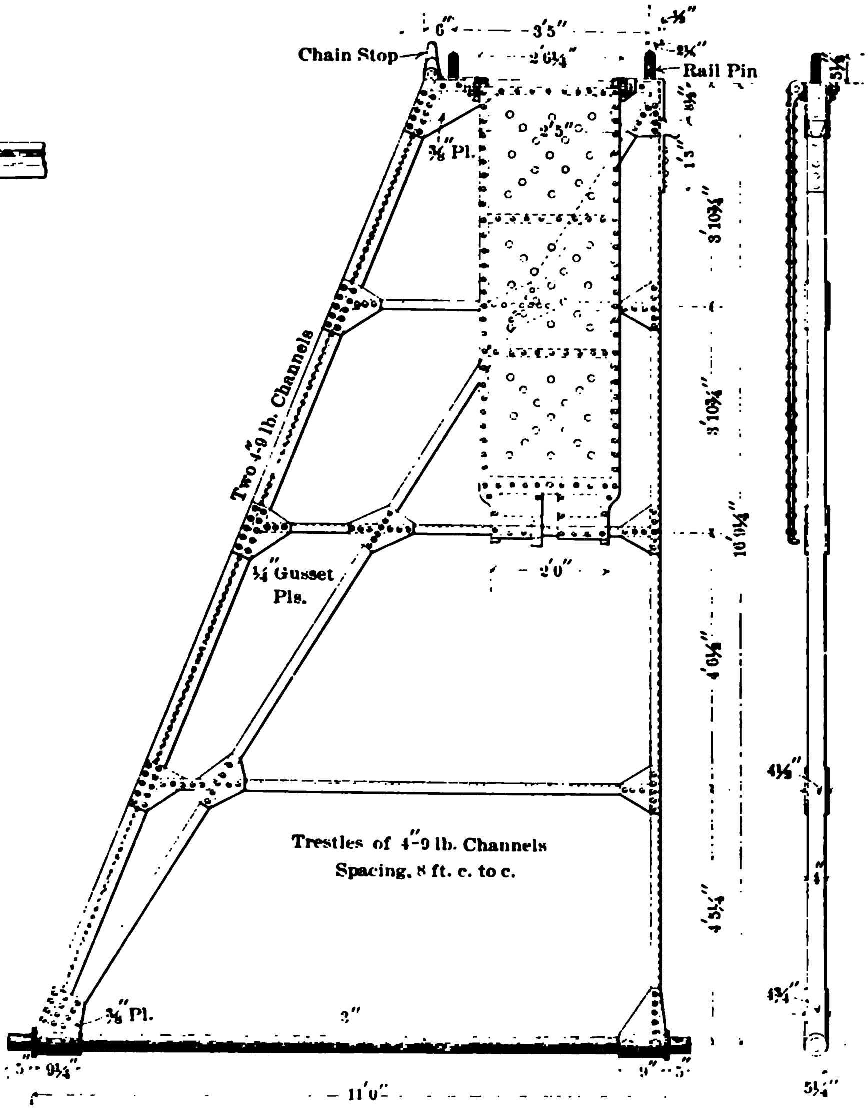




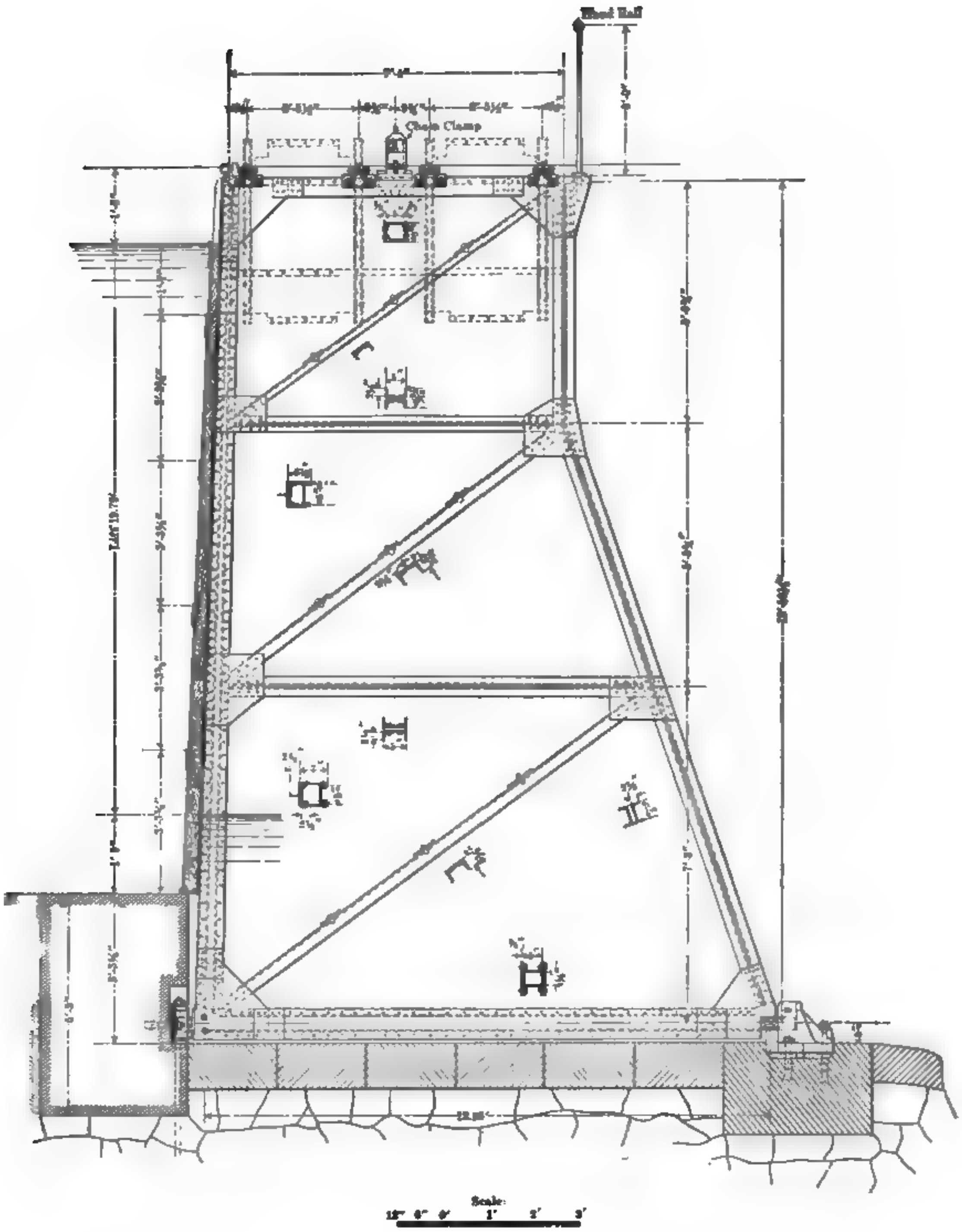
TRESTLES.

ERVICE BRIDGES,
ICKET DAMS.

le:
2' 3' 4'



PASS TRESTLE, KANAWHA RIVER. W. VA.
(1896.)

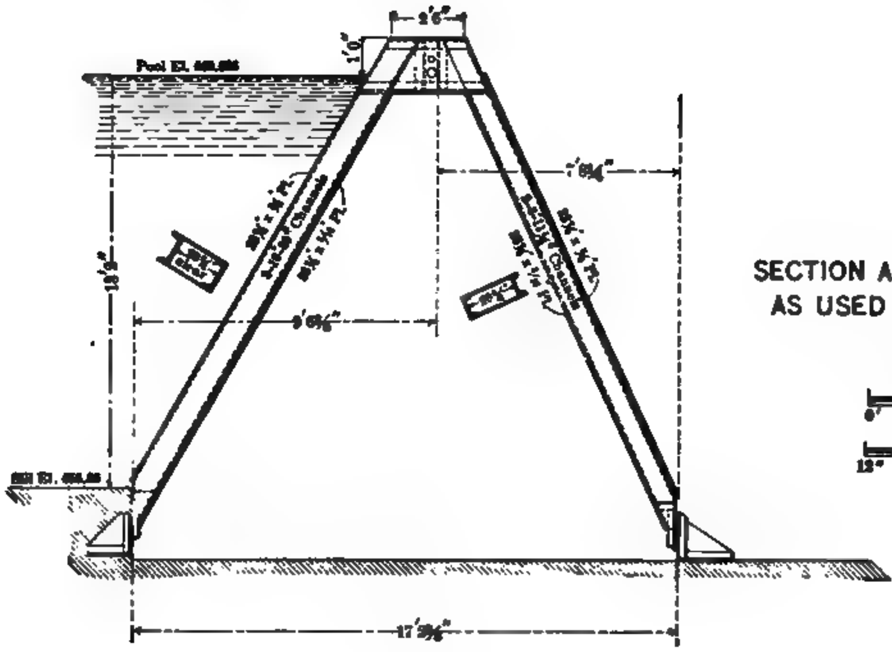


General Section of the Boulé Dam at Libschitz, Bohemia, 1900.

SECTION AND DETAILS OF A-FRAME DAM AS USED AT DAM No. 6, OHIO RIVER.

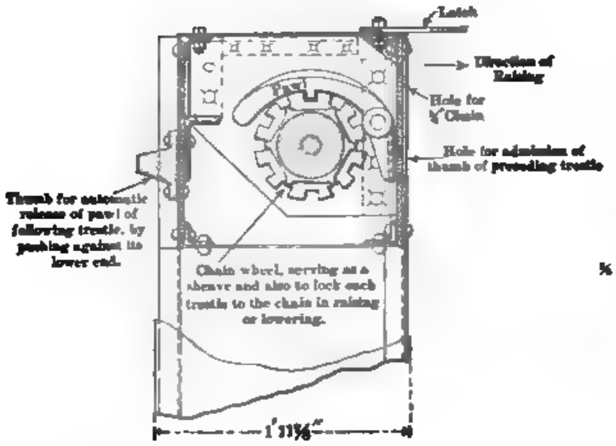
(1902.)

Scales:

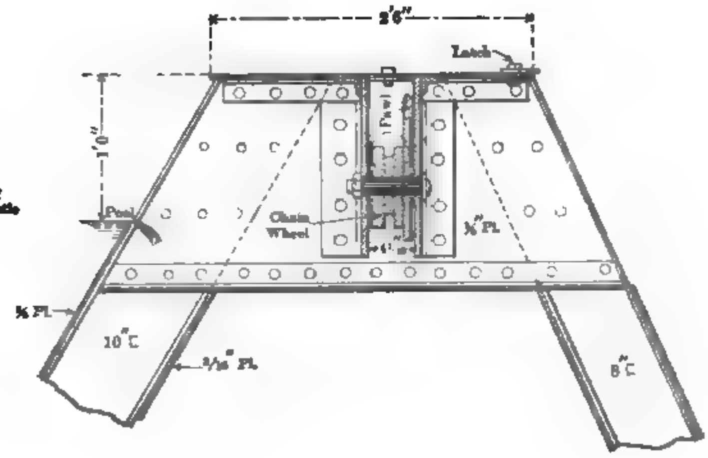


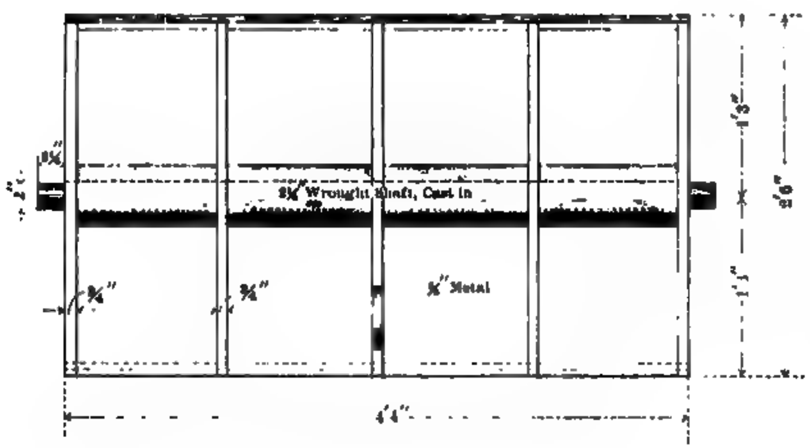
GENERAL SECTION.

SECTION OF HEAD, LOOKING DOWNSTREAM

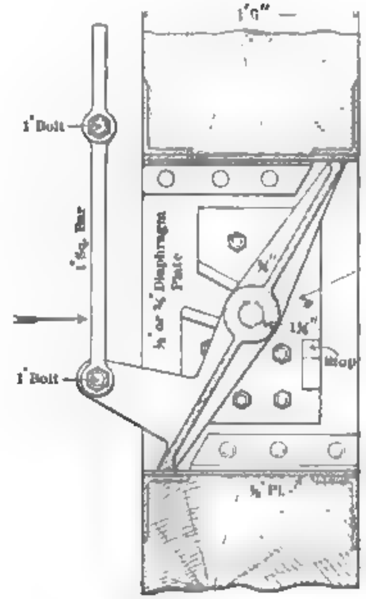


SECTION OF HEAD, LOOKING IN DIRECTION OF LOWERING.



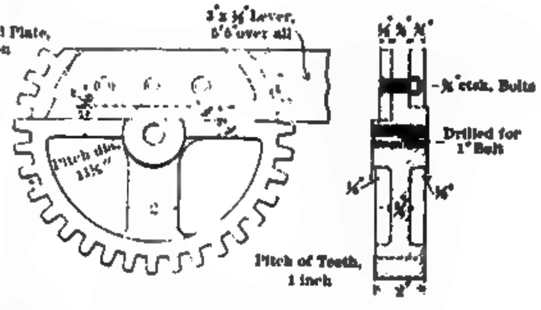


ELEVATION OF CAST IRON VALVE.



SECTION OF WOODEN GATE, SHOWING VALVE AND GIRDER IN PLACE.

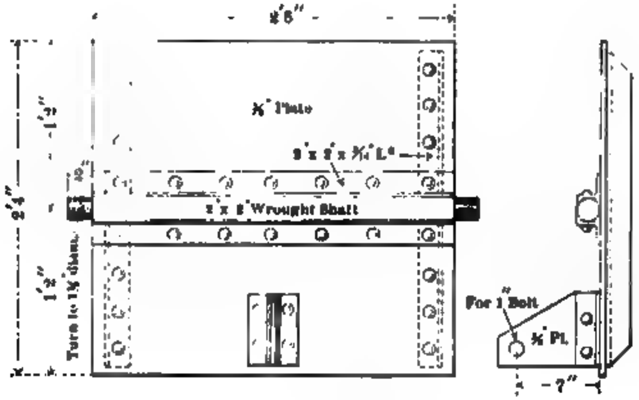
This style of valve is also used in a reversed position, with operating rods arranged to push down for opening, instead of pulling up as shown.



ELEVATION.

SECTION.

DETAILS OF SPUR-WHEEL, used in connection with Rack, for operating Gate Valves.

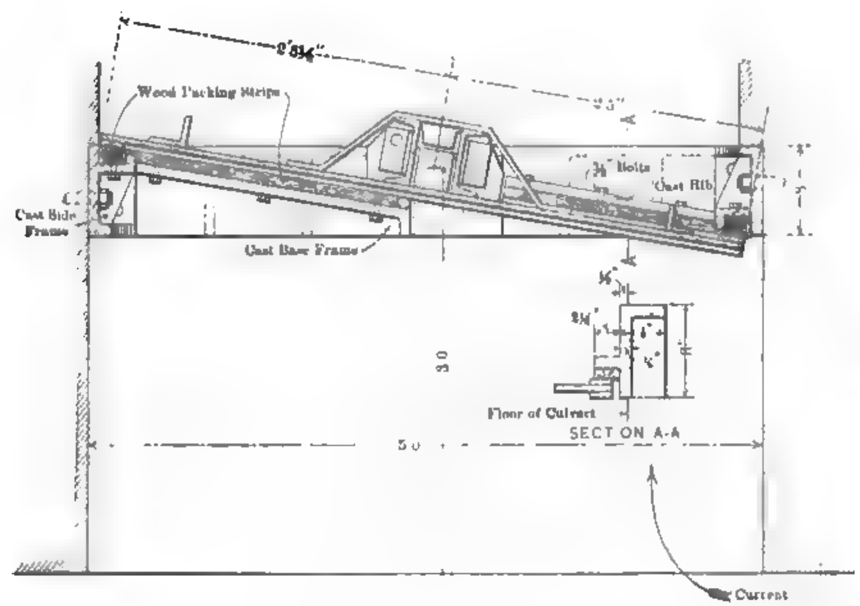


ELEVATIONS OF STEEL VALVE. (of size for high lifts.)

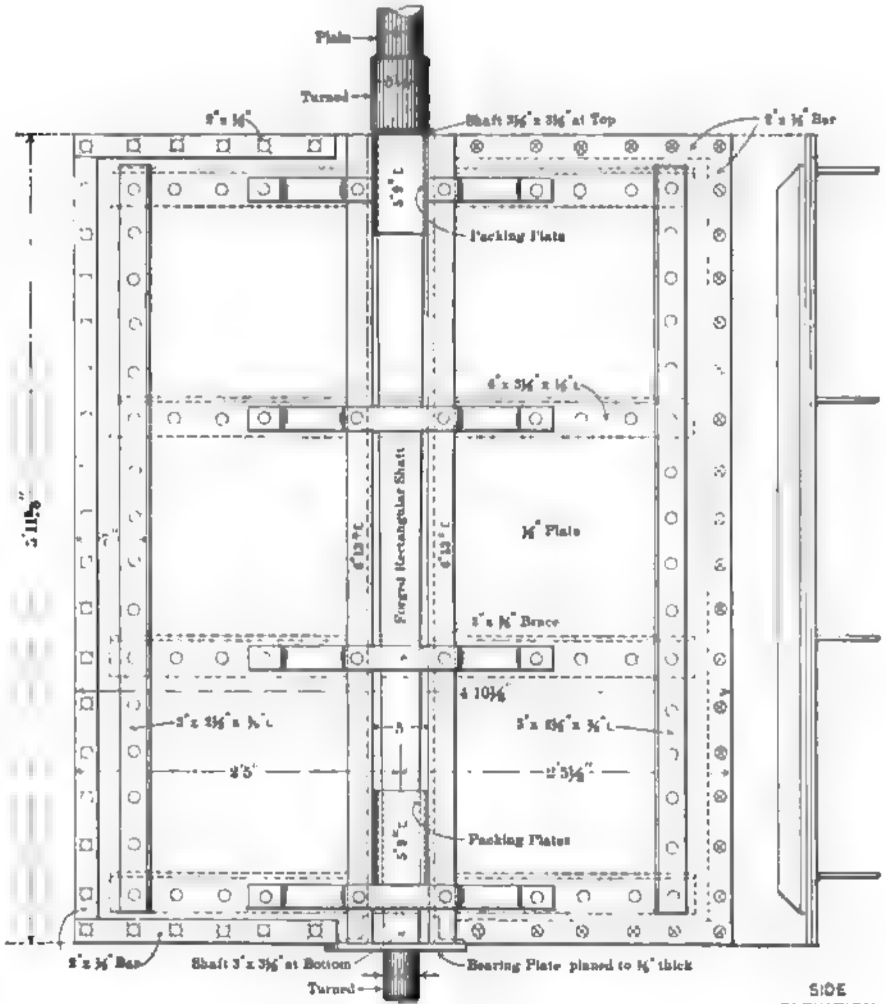
END ELEVATION OF VALVE FORMED OF TWO STEEL PLATES, BENT.

DETAILS OF BALANCED VALVES, AS USED IN LOCK-GATES.

Scales:



SECTION OF CULVERT SHOWING WICKET AND BASE AND SIDE FRAMES IN POSITION
(6" Angles omitted)



REAR ELEVATION OF WICKET SHOWING SHAFT IN PLACE.

SIDE ELEVATION

BALANCED WICKETS OF STEEL, FOR CULVERTS, AS USED ON GREEN AND KENTUCKY RIVERS, KY.
(1897-1900.)

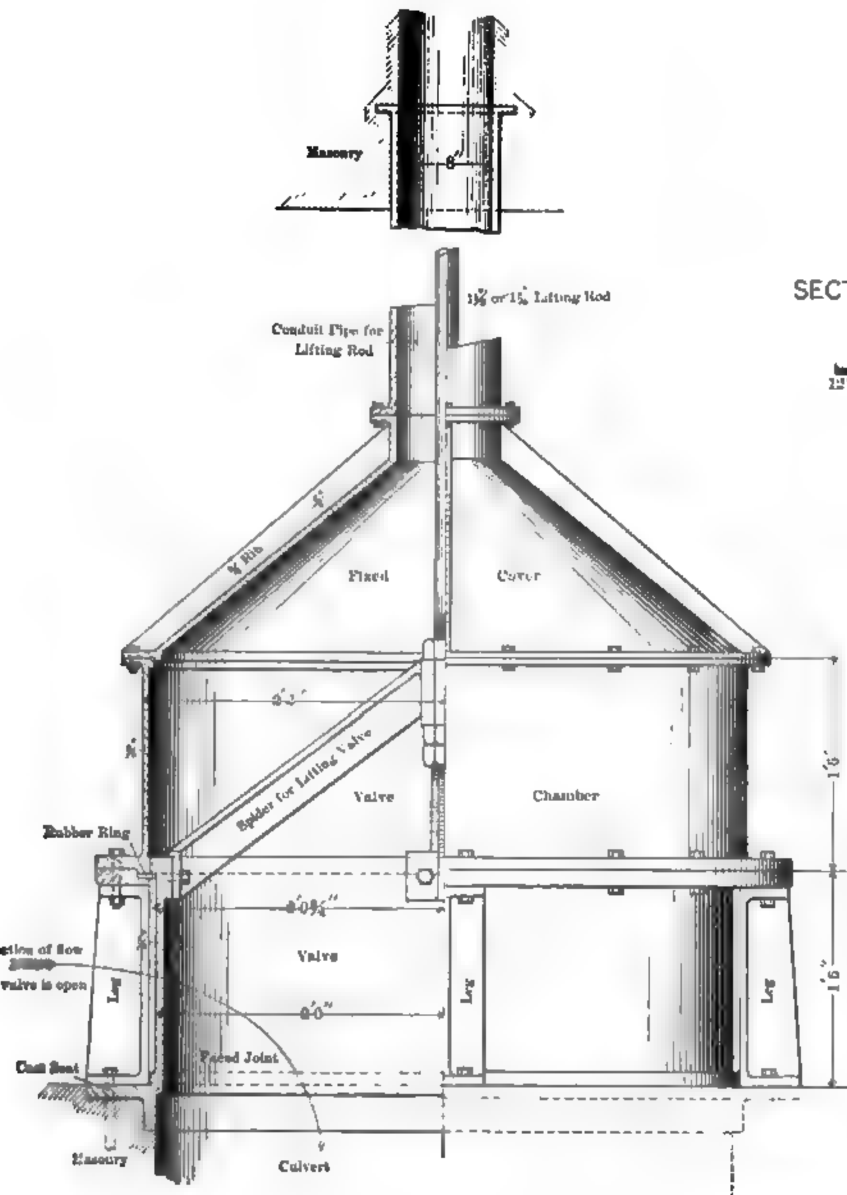
Scale

The operating machinery for these wickets consists of a train comprising two gear wheels of 39 inches pitch diameter and two pinions of $7\frac{11}{16}$ inches pitch diameter, all of $3\frac{1}{4}$-inch face and $1\frac{3}{4}$-inch pitch of teeth. For heads up to about 14 feet one man can operate a wicket without difficulty.

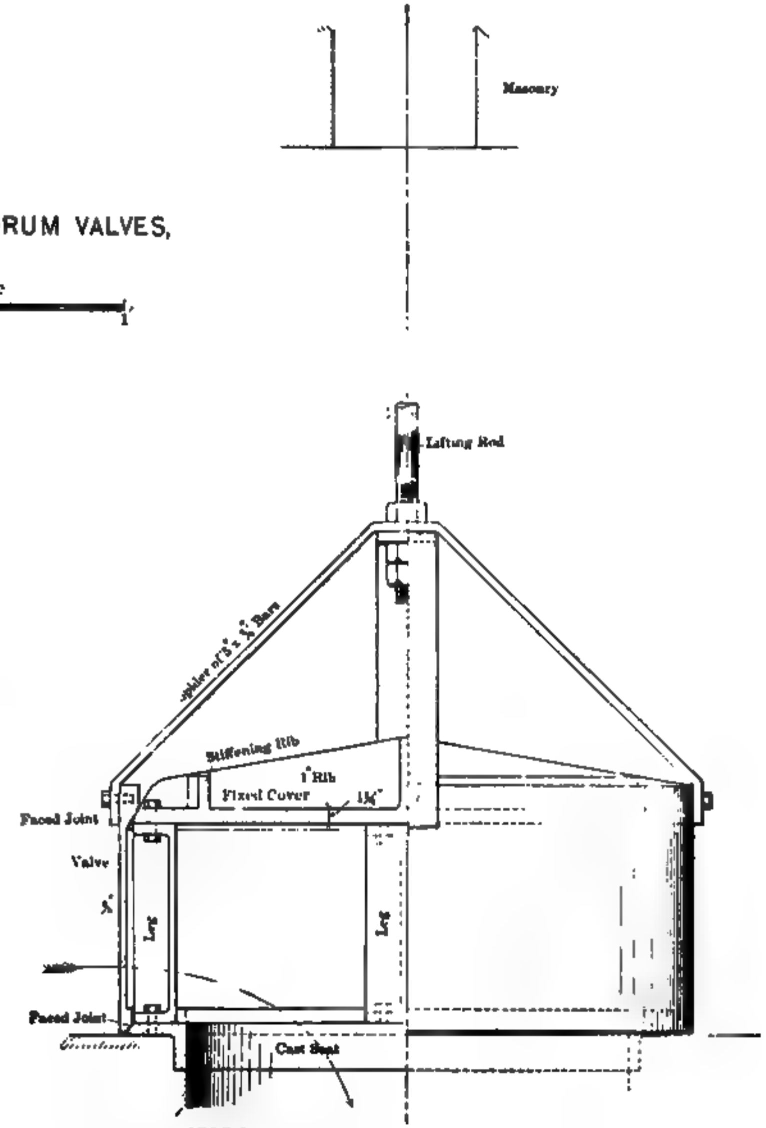
SECTIONS OF DRUM VALVES,

Scale

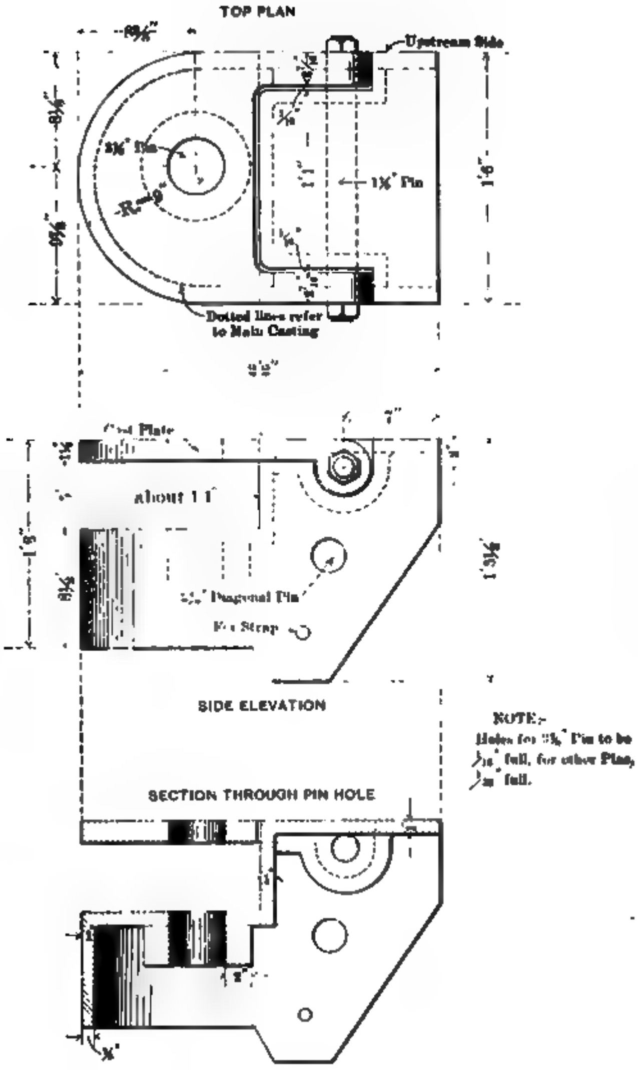
12" 9" 6" 3" 0 1'



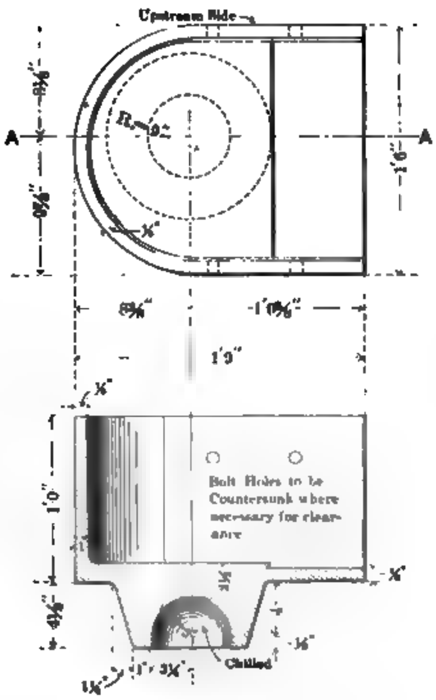
SECTIONAL ELEVATION OF DRUM OR CYLINDRICAL VALVE, FONTAINES VALVE AS USUALLY MADE, WITH VALVE MOVING IN WATERTIGHT CHAMBER.



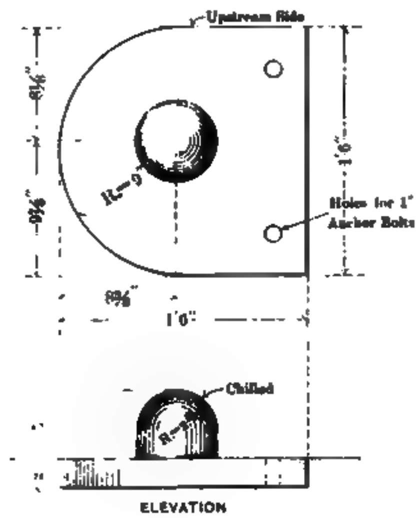
SECTIONAL ELEVATION OF PROPOSED DRUM VALVE WITHOUT WATERTIGHT CHAMBER.



CAST BONNET WITH HINGED CAP PLATE FOR AN 18-INCH GATE

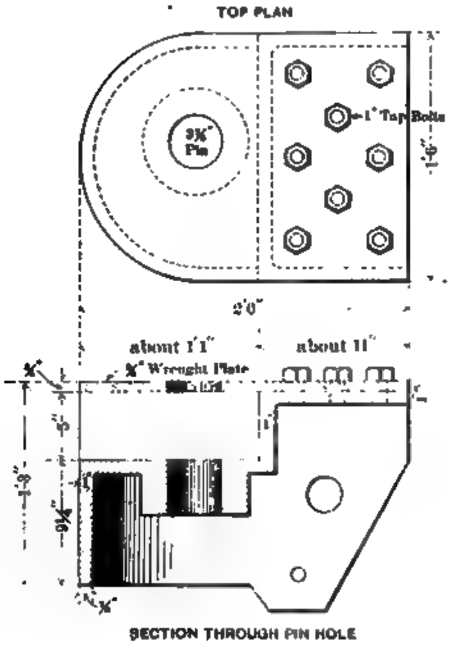


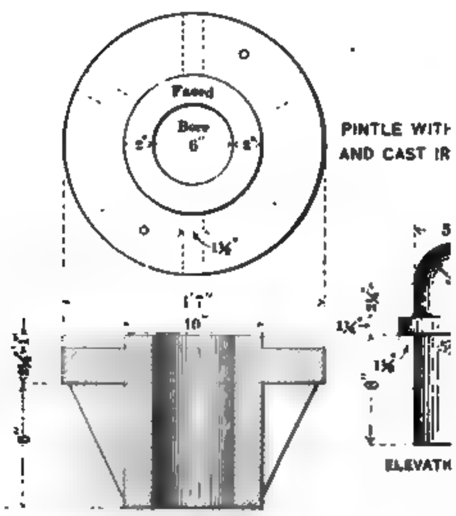
CAST SHOE FOR AN 18-INCH GATE.



PINTLE FOR AN 18-INCH GATE, IN ONE PIECE
(For a 12-inch sill)

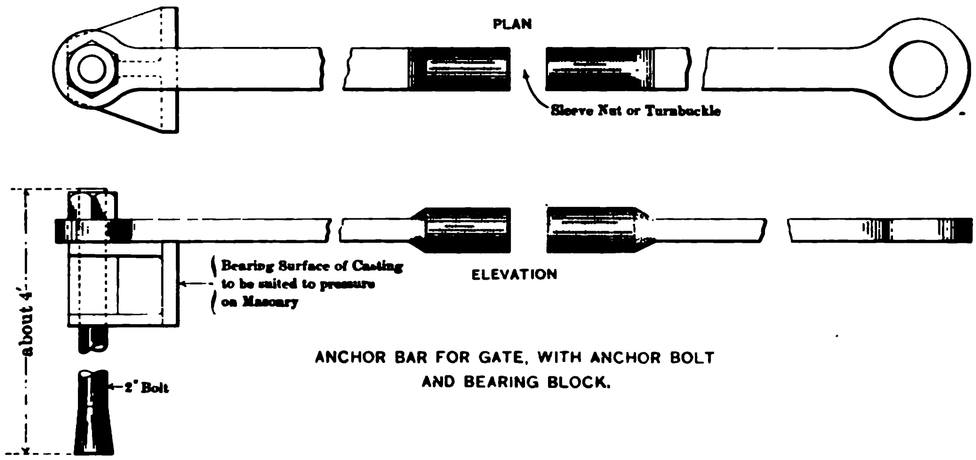
CAST BONNET WITH BOLTED CAP-PLATE.





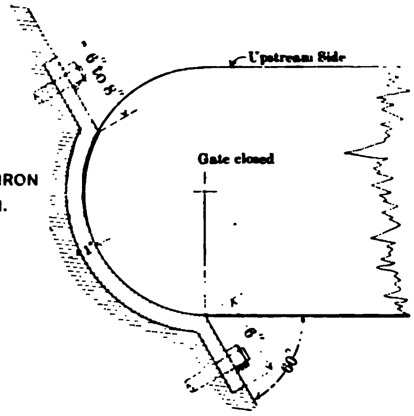
PLAN AND SECTION OF SOCKET

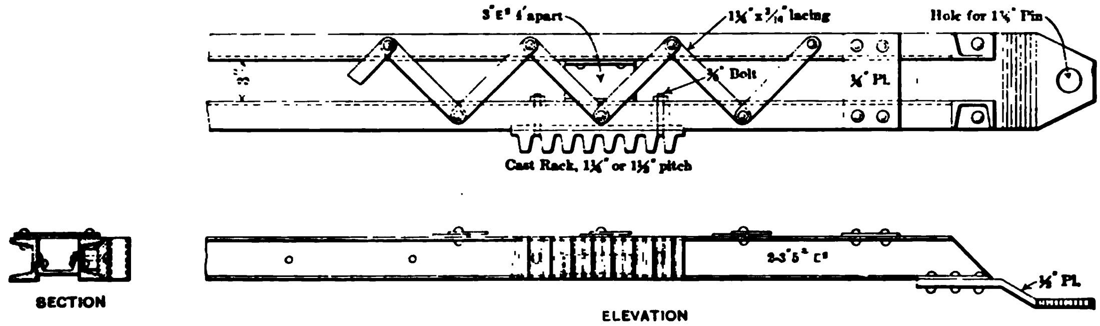
PINTLE WITH
AND CAST IR



ANCHOR BAR FOR GATE, WITH ANCHOR BOLT AND BEARING BLOCK.

SECTION OF CAST IRON HOLLOW QUOIN.





PIN
KET.

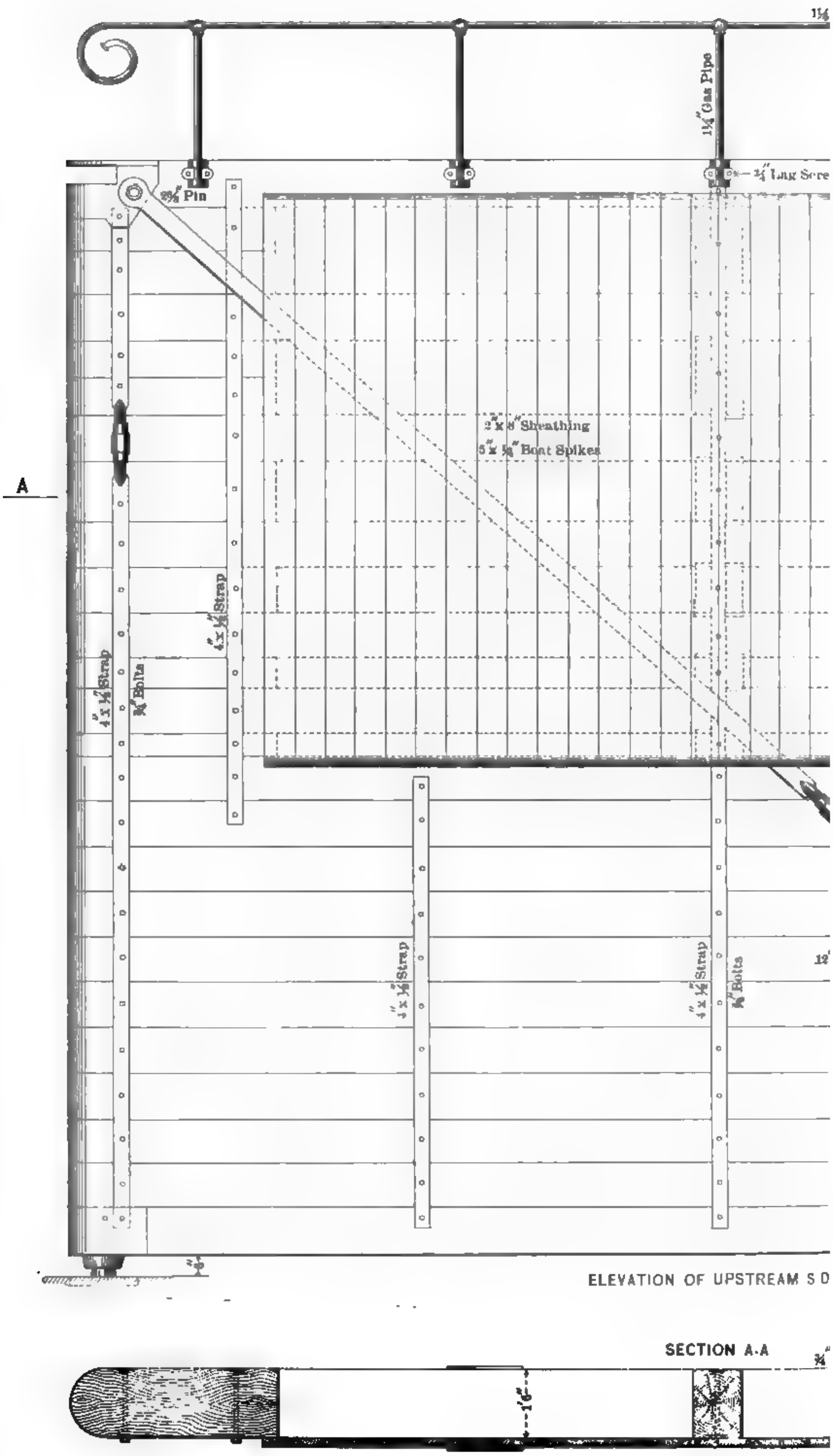
STEEL SPAR FOR OPERATING GATES.

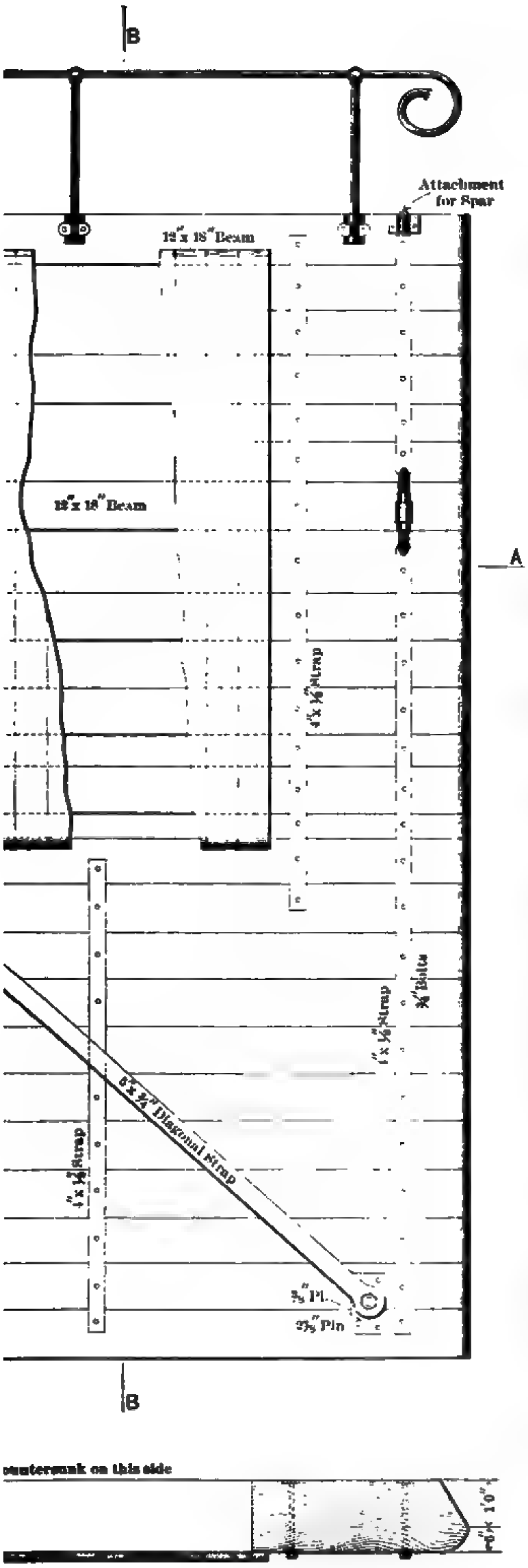
FITTINGS FOR LOCK GATES.

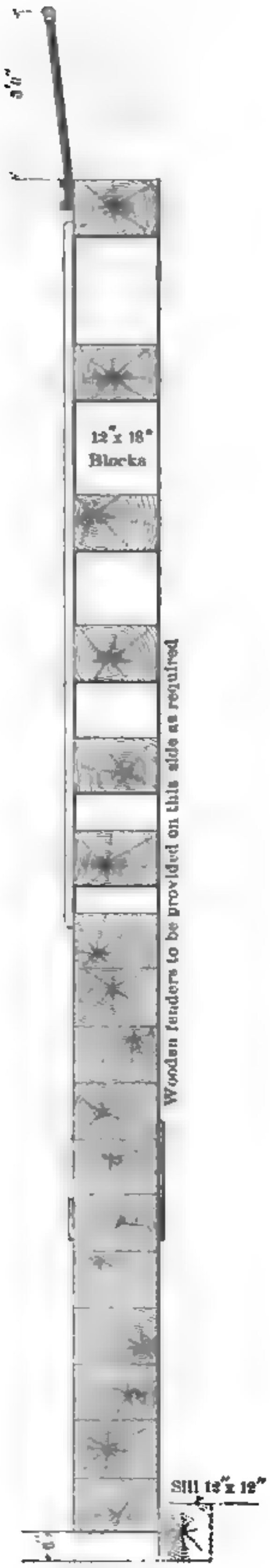
Pin.turned
all over

Scale

12" 9" 6" 3" 0' 1'


1¼
1¼" Gas Pipe
¼" Lag Scre
2⅝" Pin
2"x 8" Sheathing
5"x ½" Boat Spikes
A
4"x ½" Strap
¾" Bolts
4"x ½" Strap
4"x ½" Strap
4"x ½" Strap
¾" Bolts
12
ELEVATION OF UPSTREAM S D
SECTION A-A
¾"
1'6"





SECTION B-B

DESIGN FOR AN 18-INCH WOODEN LOCK-GATE.

Scale:

12" 6" 0' 1' 2' 3'

INDEX.